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INFORMATION RETRIEVAL

The key words, abstract, and reference "cards" for each article in this Journal represent part of the ASCE participation in the EJC information retrieval plan. The retrieval data are placed herein so that each can be cut out, placed on a 3 × 5 card and given an accession number for the user's file. The accession number is then entered on key word cards so that the user can subsequently match key words to choose the articles he wishes. Details of this program were given in an August, 1962 article in CIVIL ENGINEERING, reprints of which are available on request to ASCE headquarters.

[a]Discussion period closed for this paper. Any other discussion received during this discussion period will be published in subsequent Journals.

16646 SATELLITE POSITIONING AT SEA

KEY WORDS: **Doppler systems;** Equipment specifications; **Hydrographic surveys;** Hydrography; **Positioning; Satellites;** Surveying

ABSTRACT: A description of the present doppler satellite positioning system is given along with guidelines on its use for hydrographic surveying operations. Characteristics of several commercially available systems for vessel positioning are listed along with criterion for determining optimum equipment selection. The operational aspects of surveying in remote regions using satellite positioning is considered. Formulas for transforming coordinates from one datum to another are given.

REFERENCE: Collins, James (Vice-Pres., Craven Thompson & Assoc., Inc.), "Satellite Positioning at Sea," *Journal of the Surveying and Mapping Division,* ASCE, Vol. 107, No. SU1, **Proc. Paper 16646**, November, 1981, pp. 1-9

16623 TRANSMISSION LINE SURVEYS

KEY WORDS: Analytical photogrammetry; **Cost effectiveness;** Mapping; **Photogrammetry; Surveying;** Surveying instruments; **Transmission lines**

ABSTRACT: A fully analytical stereoplotter has been acquired by the Bonneville Power Administration to reduce the time and costs required to survey transmission lines for engineering design. Two major transmission line projects have recently been surveyed photogrammetrically using the analytical stereoplotter. Costs in terms of man-hours to perform several types of surveys by field survey crews, mechanical reconstruction stereoplotters and the analytical stereoplotter are compared and discussed. The analytical instrument has been found to be highly cost effective for specialized types of surveys requiring frequent re-setting of models of photographic stereopairs.

REFERENCE: Williams, Kirk E. (Head, Surveying and Mapping Section, Branch of Transmission Engrg., Bonneville Power Administration, Portland, Oreg.), and Wilson, Wallace C., "Transmission Line Surveys by Analytical Stereoplotter," *Journal of the Surveying and Mapping Division,* ASCE, Vol. 107, No. SU1, **Proc. Paper 16623**, November, 1981, pp. 11-20

16651 STANDARD FOR SYMBOLOGY ON ENGINEERING SCALE MAPS

KEY WORDS: **Cartography;** Design; Drawings; Mapping; **Maps; Standards; Symbols**

ABSTRACT: Though many organizations have been studying or emphasizing the need for a standard set of map symbols for large scale maps (1:240 to 1:4800), little to date has been done to develop a standardized legend. The main problems associated with producing a standard legend are, providing a unique symbol for each feature, making computer programing of the symbols possible, implementing the standard once it is adopted, and maintaining the standard. These problems and their solutions are addressed. The actual set of symbols and their development are presented in Chapter V of the forthcoming manual, "Manual on Map Uses, Scales, and Accuracies for Engineering and Associated Purposes". This manual is being prepared by the Committee on Cartographic Surveying, of the Surveying and Mapping Division, ASCE.

REFERENCE: Jacober, Robert P., Jr. (Cartographic Staff Officer, Defense Mapping Agency Aerospace Center, St. Louis, Mo. 63118), "Standard for Symbology on Engineering Scale Maps," *Journal of the Surveying and Mapping Division,* ASCE, Vol. 107, No. SU1, **Proc. Paper 16651**, November, 1981, pp. 21-24

16629 THE SURVEYING ENGINEER AND NAD-83

KEY WORDS: **Cadastral surveys;** Coordinates; Ellipsoids; **Geodetic coordinates; Pacific Northwest;** Parameters; **Surveying**

ABSTRACT: Parameters used for NAD-83 and the datum shifts expected in the Pacific Northwest are reviewed. The suggested changes needed in State laws are discussed with reference to suggestions for a model law by NGS. The current use of State Plane Coordinates by survey engineers is discussed in terms of boundary, control, and multi-purpose cadastre surveys and the expected change of technique and records are analyzed.

REFERENCE: Colcord, J. E. (Prof., Dept. of Civ. Engrg., Univ of Washington, Seattle, Wash. 98195), "The Surveying Engineer and NAD-83," *Journal of the Surveying and Mapping Division,* ASCE, Vol. 107, No. SU1, **Proc. Paper 16629**, November, 1981, pp. 25-31

16647 SURVEY CONTROL FOR I-205 COLUMBIA RIVER BRIDGE

KEY WORDS: **Bridge construction;** Columbia River; **Control joints; Distance measuring equipment; Electronic equipment; Positioning;** Surveying

ABSTRACT: The establishment of the principal control points required for the Columbia River Bridge construction using present day electronic distance measuring equipment is reviewed. Selection of the survey equipment was based on the distances at the bridge site to be measured and simplicity of operation. Familiarization and confidence were gained by traversing the river and verifying existing points set earlier by the location survey. The bridge site was studied and five locations were selected for permanent monuments. Shelters for protection of the instrument were constructed, distances measured and angles turned. Closure was made and coordinates determined with the aid of a computer terminal and the Highway Division COGO program. To date the bridge piers and some of the superstructure have been successfully positioned with a minimum effort utilizing the five control points. The electronic distance measuring equipment and computer terminal have proven to be a time and labor saving combination for establishing survey control of this type.

REFERENCE: Howard, John D. (Resident Engr., Oregon Highway Div., Dept. of Transportation, State Highway Div., 1919 NW Thurman Portland, Oreg. 97209), "Survey Control for I-205 Columbia River Bridge," *Journal of the Surveying and Mapping Division,* ASCE, Vol. 107, No. SU1, **Proc. Paper 16647**, November, 1981, pp. 33-44

16659 RETRACEMENT SURVEYS IN PACIFIC NORTHWEST

KEY WORDS: Boundaries (property); **Contracts; Estimates;** History; Land ownership; Monuments; Pacific Northwest; **Public land; Surveying**

ABSTRACT: Search and perpetuation of U.S. Public Land Survey section corners makes contracting for section subdivision surveys less costly. The U.S. Forest Service needs to identify 1,500 miles of property boundary per year. The contract estimates and statement of work needed is more realistic when based on recovered original survey corners. Forest Service personnel rely on the original instructions issued the surveyor, the manual in effect at time of survey and the knowledge gained in previous search and retracements. Attention is given to the monuments, accessories to the monument and traits of individual surveyors. Emphasis is given to evaluation, perpetuation and recordation of evidence. Corner information is filed with the Forest Supervisor and is available to the public. The data provides the information for identifying the forest boundaries and determining the surveys needed for the Land Adjustment Program.

REFERENCE: Long, Gordon H. (Regional Cadastral Engr., U.S. Forest Service, Region Six, Portland, Oreg.), "Retracement Surveys in Pacific Northwest Coast Range," *Journal of the Surveying and Mapping Division,* ASCE, Vol. 107, No. SU1, **Proc. Paper 16659**, November, 1981, pp. 45-58

16672 GEOMETRIC FRAMEWORK FOR LAND DATA SYSTEMS

KEY WORDS: **Cadastral surveys;** Computer applications; **Data systems;** Earth sciences; **Land classification;** Land titles; **Land usage planning;** Mapping; Surveying

ABSTRACT: The necessary geometric framework for the successful development and application of such land data systems should permit identification of land areas by coordinates down to the individual parcel level, while permitting the precise mathematical correlation of real property boundary and earth science data. The paper examines such a geometric framework developed by combining two unrelated survey control systems established by the federal government for real property boundary and earth science mapping. Although the specific geometric framework described is applicable only to those parts of the United States which have been covered by the U.S. Public Land Survey System, the fundamental concept involved is applicable to any area.

REFERENCE: Bauer, Kurt W. (Executive Dir., Southeastern Wisconsin Regional Planning Commission, Waukesha, Wisc. 53187), "Geometric Framework for Land Data Systems," *Journal of the Surveying and Mapping Division,* ASCE, Vol. 107, No. SU1, **Proc. Paper 16672**, November, 1981, pp. 59-65

U.S. Customary-SI Conversion Factors

In accordance with the October, 1970 action of the ASCE Board of Direction, which stated that all publications of the Society should list all measurements in both U.S. Customary and SI (International System) units, the following list contains conversion factors to enable readers to compute the SI unit values of measurements. A complete guide to the SI system and its use has been published by the American Society for Testing and Materials. Copies of this publication (ASTM E-380) can be purchased from ASCE at a price of $3.00 each; orders must be prepaid.

All authors of *Journal* papers are being asked to prepare their papers in this dual-unit format. To provide preliminary assistance to authors, the following list of conversion factors and guides are recommended by the ASCE Committee on Metrication.

To convert	To	Multiply by
inches (in.)	millimeters (mm)	25.4
feet (ft)	meters (m)	0.305
yards (yd)	meters (m)	0.914
miles (miles)	kilometers (km)	1.61
square inches (sq in.)	square millimeters (mm^2)	645
square feet (sq ft)	square meters (m^2)	0.093
square yards (sq yd)	square meters (m^2)	0.836
square miles (sq miles)	square kilometers (km^2)	2.59
acres (acre)	hectares (ha)	0.405
cubic inches (cu in.)	cubic millimeters (mm^3)	16,400
cubic feet (cu ft)	cubic meters (m^3)	0.028
cubic yards (cu yd)	cubic meters (m^3)	0.765
pounds (lb) mass	kilograms (kg)	0.453
tons (ton) mass	kilograms (kg)	907
pound force (lbf)	newtons (N)	4.45
kilogram force (kgf)	newtons (N)	9.81
pounds per square foot (psf)	pascals (Pa)	47.9
pounds per square inch (psi)	kilopascals (kPa)	6.89
U.S. gallons (gal)	liters (L)	3.79
acre-feet (acre-ft)	cubic meters (m^3)	1,233

Satellite Positioning at Sea [a]

By James Collins,[1] M. ASCE

Introduction

The precise positioning of vessels beyond a few hundred kilometers from land has presented a seemingly insolvable problem until the advent of satellite positioning. Beginning in the early 1960s, the United States began an extensive program to develop navigation satellite systems for defense purposes. In 1967 commercial applications of U.S. Navy TRANSIT navigational satellites became possible, and the use of this system has steadily increased since this time. Although there have been other satellite positioning systems, the TRANSIT system has been in general continuous use since 1967 (16), and in this chapter will be referred to as the satellite system.

The commercial user of satellite positioning does not often concern himself with the basic theory of its operation. Most users purchase a positioning system which includes the computer programs necessary to determine a geodetic position from the observed satellite data. One should be aware, however, of the general principle of operation of satellite positioning systems so that a clearer understanding of their limitations results.

There are currently five operational satellites in polar orbits approx 1,000 km above the Earth's surface with an orbital period of approx 100 min. These five satellites are tracked by four ground stations (Hawaii, California, Minnesota, Maine) whose geodetic coordinates have been precisely determined (13). The best-fit satellite orbits are computed for each satellite from the tracking data observed from the four fixed ground stations. This orbital information is periodically broadcast to the satellites which then rebroadcasts the information to the user. The orbital information broadcast by the satellite every 2 min is basically equivalent to the spatial coordinates (X, Y, Z) of the satellite at the instant of the broadcast (8).

[a] Presented at the October 22–26, 1979, ASCE Annual Convention & Exposition, held at Atlanta, Ga. (Preprint 80-3624).

[1] Pres., G.P.H. Consultants, 414 Hungerford Drive, Suite 200, Rockville, Md. 20850.

Note.—Discussion open until April 1, 1982. To extend the closing date one month, a written request must be filed with the Manager of Technical and Professional Publications, ASCE. Manuscript was submitted for review for possible publication on November 5, 1979. This paper is part of the Journal of the Surveying and Mapping Division, Proceedings of the American Society of Civil Engineers, ©ASCE, Vol. 107, No. SU1, November, 1981. ISSN 0569-8073/81/0001-0001/$01.00.

Satellite Observations

The position of the observer is computed by measuring the range or difference in ranges to the satellite as it passes overhead. Since the spatial coordinates of the satellite are known, the observer's position can be computed from the range differences—a method similar to the computation of position from Loran range difference data. The observable or measured quantity is actually the Doppler frequency as the satellite passes overhead. The satellite transmits two frequencies, 400 MHz and 150 MHz, which are used to make correction for the ionospheric effects on the transmitted signal.

Doppler shifts are measured by mixing the signals received from the satellite with a ground receiver reference frequency of 400 MHz. The slight difference between the satellite and reference frequencies creates a beat frequency which is the actual quantity counted. The number of beat counts received in the fixed 2-min interval (or shorter subinterval) represents an observation. The Doppler observation equation is of the form (15):

$$(r_2 - r_1) = \Lambda\,[N - (f_r - f_t)(t_2 - t_1)] \quad \text{. . . . .} \quad (1)$$

in which r = range; f_r = reference frequency of observer; f_t = transmitted frequency of satellite; t = time at points 1 and 2, respectively; Λ = broadcast wavelength; and N = Doppler count.

The quantities on the right side of this equation are known or measured so that the difference in range between the satellite at positions 1 and 2 can be determined.

Corrections

The Doppler count (N) in the observation equation is the corrected, or ideal, count. Both tropospheric and ionospheric corrections are applied to the measured count to correct for refraction effects. The ionospheric correction is determined by intercomparing the satellite's 400-MHz and 150-MHz transmitted frequencies; the tropospheric refraction is determined by applying measured atmospheric constants. There are also a number of other minor corrections that are applied to observations requiring the maximum precision: special relativity, local clock synchronization, and receiver delay (15).

Types of Satellite Positioning

This section primarily explains satellite positioning at sea; however, other uses of satellite positioning are also described since they relate to survey operations.

Point Positioning with Doppler satellite receivers is accomplished by occupying a survey point for one or more passes of satellites. Recent improvements in the orbital data of the navigation satellites make possible the location of points to 30-m accuracy with 10 passes (7), and 3 m with 35 passes (5). These results compare closely with the point positioning using one satellite and a precise (post) ephemeris generated by tracking from 20 stations around the world (1). Global point positioning using the broadcast rather than the precise post ephemeris probably introduces errors larger than the 3 m cited due to uncertainties in

the orbit over large portions of the Earth. The 3-m accuracy achieved using 35 satellite passes was for points in the continental United States where there is a good distribution of tracking stations.

Relative Positioning Techniques is the name used for positioning points when more than one satellite receiver is used concurrently. One receiver is positioned over a point of known (or assumed) position and the second receiver over a point to be established. The relative positioning accuracies achieved with translocation are slightly better than the interstation results obtained with the point positioning (6). Equipment and computer programs are available to compute position data in the field as soon as the required number of satellite passes is achieved (5,7).

Another version of the relative positioning technique is the short arc method (4). This method uses the translocation concept of one fixed receiver and one or more mobile receivers to position unknown points. When a special computer program SAGA III or GEODOP (12) is used to reduce the data, relative accuracies (all accuracy statements are at the 3σ level) of 0.75 meters have been demonstrated using 40 satellite passes (4). Relative positioning accuracies of 3 meters can be achieved with as few as six satellite passes using this technique.

Positioning of Offshore Structures has been successfully accomplished using both point positioning (NGS) and translocation (9) techniques. In this particular application speed is generally essential so that the translocation technique has only a slight advantage over the point location technique.

Problems in the received signal have been experienced by National Geodetic Survey personnel occupying offshore structures when the Doppler antenna is located high on the structure. The best location has been proven to be directly on the main deck of the platform. Evidently the structure deck forms a suitable ground plane for the antenna which is lacking when the antenna is elevated from the main deck (9). The position accuracy can be checked by measuring down from the antenna to the water surface. After correcting for tides this height should agree with the satellite determined height (after geoidal separation correction).

Location of Electronic Shore Control Points by satellite positioning is particularly cost-effective when local geodetic control is at a distance from the site selected. In this particular application point positioning is generally advantageous since the receiver can be left running while personnel are installing the electronic positioning equipment (5). The same receiver that is used on board ship can be used to position shore control points if the receiver is equipped with an accurate time standard.

Vessel Positioning uses only a single satellite pass and computes the latitude and longitude of the vessel since the height is already precisely known (the height of the antenna above the sea surface should be carefully measured). Computations are preformed by shipboard computer with results available shortly after the satellite pass. Accuracies of ± 100 meters are generally achievable using the latest broadcast ephemeris (11). On a global average, position information is available about every 2 hr. In higher latitudes much more frequent fixes are achieved.

A number of techniques can be used to D.R. (deduced reconing) a vessel between satellite fixes. Loran C and Omega systems are two long-range navigation systems which provide reliable relative precision for determining the intermediate

points between satellite fixes. More expensive inertial navigation systems are also available for interpolating between points of known position.

A combination of systems would provide the ultimate navigation system for precise bathymetric surveying. Components of such a system have been described by McCloskey and Evans (14) to include a satellite navigator, Omega receiver, electromagnetic log, gyro compass, and an acoustic transponder system. All of these components can be combined in a post-processing mathematical mode to give after-the-fact best-fit ship positions. Bottom acoustic transponders can be located approximately by satellite fix when they are deployed. Subsequent to this, range data to the transponders are measured whenever the vessel is within range; this range and its associated position data are then stored for future processing. In this manner data for a least square solution of transponder coordinates are obtained during the normal shipboard surveying operations. This technique saves the time spent steering in a geometric pattern to locate the transponders at the time of deployment.

Satellite Navigation Equipment

There are numerous commercial satellite navigating systems available. A brief description of some of these systems is given to provide a general concept of what is presently available. These systems change (improve) rapidly, and a user wishing to purchase a system should make his own thorough market analysis before investing in a costly satellite navigator.

Autonav 1050.—Mfg:

Automated Marine International
1641 McGraw Avenue
Irving, CA 92714

This system includes, in addition to basic satellite navigation receiver, options for dual channel receivers and an integrated computer. Peripheral input-output equipment includes magnetic and paper tape, CRT, plotter, printer, sonar, gyro, and external positioning systems. Features include a Kalman filter for optimal position and velocity estimation. Physical description: 3 racks; 1,000 lb; 5 KVA; HP 21 MX computer.

CMA-722.—Mfg:

Canadian Marconi Co.
2442 Trenton Avenue
Montreal, Canada H3P 1YP

This system includes, in addition to basic satellite receiver, the option for expansion into an integrated positioning system. Peripheral equipment options include: (1) Teletype; (2) gyro compass; (3) log; (4) Doppler sonar; (5) tape reader; (6) tape punch; and (7) external navigation inputs. Physical description: 1 rack; 1 KVA; HP 21 MX computer.

JMR.—Mfg:

JMR Instruments, Inc.
20621 Plummer Street
Chatsworth, CA 91311

This system comes in both portable and fixed location configurations. Mobile sets operate on 12 volts and are small and light enough to be back packed. Another configuration offers remote unattended operation. The JMR will tailor

equipment and computer programs to meet individual customer requirements. Physical description of basic receiver: 20 Kg; 240 mm × 530 mm × 390 mm; 12 V, 4.5 Watts.

MX 1102.—Mfg:

Magnavox
Advanced Products Division
2829 Maricopa Street
Torrance, CA 90503

The 1102 Satellite Navigator is a compact integrated system that can be bulkhead or overhead mounted. The set is engineered for ease of operation and includes malfunction diagnostics and programmed tracking of selected satellites. Magnavox also manufactures satellite receivers designed specifically for geodetic positioning such as the Geoceiver II and MX 1502. Physical description MX 1102: 430 mm × 360 mm × 420 mm; 4 Kg; 150 Watts.

From the preceding descriptions, it can be seen that numerous options, configurations, and specifications confront the purchaser of satellite positioning equipment. The basic requirements of the task will help the purchaser select the optimum equipment since some configurations are tailored to specific tasks. For example, if the geodetic coordinates of control points are required, a configuration which includes the option of portable receiver operation should be selected. Furthermore, a precise clock should be included in the portable geodetic subunit of the satellite equipment.

When a fully integrated shipboard surveying system is needed, one of the system configurations which includes a post processing and least square adjustment of all input data should be selected. In the future, more advanced computers will make the use of near real time post processing more readily available.

Operational Planning

The planning of any hydrographic project is critical to efficient operation. The integrated system satellite navigational equipment previously described should be calibrated like any measuring equipment. In addition to calibrating fathometers, gyrocompass, speed log, and associated data input, the satellite receiving equipment should be checked.

When two receivers are available, one method is to intercompare computed latitudes and longitudes and heights from the same satellite pass. These computed positions should agree within the manufacturer's stated accuracy. Either separate physical locations or one antenna feeding two receivers will provide the required information. The computed geodetic coordinates can be compared with known geodetic coordinates when only one receiver is available when the proper coordinate translations are available. This can be accomplished at the dock or by intercomparing a fix from a more accurate system. In any case these checks will indicate the presence of bias in the satellite equipment.

The best procedure to follow is to occupy a geodetic control point with known X, Y, Z coordinates. The satellite positioning equipment will give accurate satellite derived X, Y, Z coordinates of this same point after observing approx 40 passes. The algebraic difference between these two sets of coordinates will give the ΔX, ΔY, ΔZ translations values necessary to transform satellite coordinates to coordinates of a particular datum. A periodic remeasurement of the X, Y,

Z coordinates of the datum point will give an indication of the satellite's receivers proper operation.

When vessels are operating beyond the reach of accurate electronic positioning system (hi-fix, Raydist) bottom transponders can be used to provide recoverable point positions. As previously mentioned the coordinates of bottom transponders can be determined from a least square solution after a sufficient number of satellite fixes and transponders ranges have been observed. Navigational satellite fixes converge quite rapidly, with approx 40 passes providing the "true" position. Bullseye plots of fixes taken over a 6-month period show approx 90% of the computed positions fall within 0.05 nmi of the mean position (Ref. 11, pg. 24).

Satellite receivers can be used to establish the coordinates of fixed control points when they are required in conjunction with the hydrographic surveys. Current results indicate that the relative positioning technique is slightly preferable to the point positioning mode of operation. Satellite receivers used to establish control points should be checked in a manner similar to the shipboard equipment. Both sets operated at the same physical (geographic) point should give the same results for the same satellite pass. Small differences can be expected due to the electronic differences between the sets. Once it has been confirmed there are no gross errors or differences between the two sets, one set is operated continuously at a known control point and the second set is moved from one point to another. Tests show that an accuracy of ± 3 m is achievable by observing 35 satellite passes (5). The equipment used in this particular test was JMR automated equipment that gathered the data in an unattended mode. In this unattended mode the position of a shore site can be determined while personnel are engaged in other work activity.

Datums

Computations of positions using Doppler satellite data are made in a three-dimensional Earth centered coordinate system. A detailed description of this coordinate system and datum transformations is given in Defense Mapping Agency (DMA) Technical Manual T-3-52320, dated November 1976. This manual, entitled Satellite Records Manual, Doppler Geodetic Point Positioning, can be obtained from the DMA, Washington, D.C. 20305.

The precise ephemeris generated by tracking one of the navigation satellites for use in geodetic positioning is in a coordinate system called the World Geodetic System—1972 or WGS-72. This system uses a highly refined gravity model with spherical harmonic terms to J_{20}^{20} to describe the earth's gravity field (2). Geodetic latitudes and longitudes are generally expressed in values which refer to a reference ellipsoid with a semimajor axis (a) 6,378,135 m and a reciprocal of flattening (f) of 1/298.26. The position values derived from observing Doppler satellites are expressed in X, Y, Z coordinate values, however, and can be readily transformed to ϕ, λ, h values referenced to any ellipsoid by approximate formulas (given in the following section).

The coordinate system associated with the navigation satellites broadcasting the ephemeris is not currently identical to the WGS-72 system. Discrepancies of up to 10 m in position can occur between the precise geodetic satellite positions and positions derived from the navigation satellite. An attempt is presently

being made to bring these two systems into agreement (10). The reference ellipsoid and gravity model associated with the navigation satellites is the same as the values used for the precise geodetic satellite.

Confusion as to which system is referred to is eliminated if the surveyor establishes his own ΔX, ΔX, ΔZ values by occupying a point of known geodetic position. These coordinate origin shifts can then be applied to the satellite derived X, Y, Z values and the ϕ, λ, h values are then computed from these shifted X, Y, Z coordinates.

For example the mean difference between the coordinate system origins for the North American Datum—27 (NAD-27) system and WGS-72 is given as: (17)

$\Delta X = -22\ m$; $\Delta Y = 157\ M$; $\Delta Z = 176\ m$

In which Δ = (WGS-72) − (NAD-27)

Thus to obtain X, Y, Z NAD-27 coordinates from WGS-72 coordinates:

$$\begin{bmatrix} X \\ Y \\ Z \end{bmatrix}_{\text{NAD-27}} = \begin{bmatrix} X \\ Y \\ Z \end{bmatrix}_{\text{WGS-72}} + \begin{bmatrix} +22 \\ -157 \\ -176 \end{bmatrix} \quad \text{(2)}$$

After applying this shift the latitude, longitude and height of any point can be computed from the following formulas:

Longitude (λ) + West

Latitude (ϕ) + North

$$e^2 = \frac{a^2 - b^2}{a^2} \quad \text{(3)}$$

$$N = a(1 - e^2 \sin^2 \phi)^{-1/2} \quad \text{(4)}$$

$$X = (N + h) \cos \phi \cos \lambda \quad \text{(5)}$$

$$Y = -(N + h) \cos \phi \sin \lambda \quad \text{(6)}$$

$$Z = [N(1 - e^2) + h] \sin \phi \quad \text{(7)}$$

Conversion of X, Y, Z to ϕ, λ, h is made by:

$$\tan \lambda = \frac{-Y}{X} \quad \text{(8)}$$

$$\tan \phi = \frac{\dfrac{Z}{(1 - e^2)} - \dfrac{h}{(1 - e^2)} \sin\phi}{\dfrac{-Y}{\sin \lambda} - h \cos \phi} \quad \text{(9)}$$

$$\text{in which} \quad h = \frac{\dfrac{-Y}{\sin \lambda}}{\cos \phi} - N \quad \text{(10)}$$

These formulas are iterative in that an approximate latitude is computed from the first term of the tangent ϕ function. This approximate latitude is then used to compute the approximate elevation h and the small terms including (h) and (ϕ) in the tangent function. A corrected final latitude can generally be computed after the height terms in the tan ϕ formula have been made. When observations are made at sea level as on a ship, the value of h will generally be small.

Future Systems

The Federal Government plans to deploy a new satellite positioning system by the mid-1980s. This system is called the NAVSTAR Global Positioning System (3) and will eventually use 18 satellites, six in each of three orbital planes, separated by 120°. The satellite will circle the earth every 12 h and broadcast signals on two frequencies, 1,575.42 MHz and 1,227.6 MHz. When it is operational, the NAVSTAR system should provide dynamic accuracies of about 10 m and point position accuracies of less than a meter. Present plans call for a variety of receiver equipment spanning the scale from simple to complex. The program NAVSTAR will give continuous navigation position information directly without resorting to the use of an auxiliary system such as a Loran C.

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Appendix II.—Notation

The following symbols are used in this paper:

a = earth's reference ellipsoid semimajor axis;
b = earth's reference ellipsoid semiminor axis;
e = eccentricity of earth's reference ellipsoid;
f = frequency;
h = height above earth's reference ellipsoid;
N = radius of curvature in prime vertical of earth's reference ellipsoid;
n = Doppler count;
r = range or distance;
t = time;
X = axis of Universal Space Coordinate with origin at earth's center and passing through 0° latitude and 0° longitude;
Y = axis of Universal Space Coordinate with origin at earth's center and passing through 90° E longitude;
Z = axis of Universal Space Coordinate with origin at earth's center and passing through North Pole;
Λ = broadcast wavelength;
λ = geodetic longitude (0° through Greenwich);
ϕ = geodetic latitude (0° at equator); and
σ = standard error level of error.

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Transmission Line Surveys by Analytical Stereoplotter[a]

By Kirk E. Williams[1] and Wallace C. Wilson[2]

Introduction

Photogrammetry has been developed as a tool for surveying for all types of mapping and engineering design by the surveying and mapping section of the Bonneville Power Administration over the past 15 yr. Specialized needs for the comparison of alternate routes, from the standpoints of cost and environmental analysis, the design of transmission lines and substations, and the minimization of right-of-way clearing have been the primary concerns in this development. However, needs for timeliness in design of properties on which survey crews could not enter have added to the impetus of the program. Court suits filed by individuals, groups claiming a lack of compliance with the Environmental Policy Act, and extended negotiations with property owners prior to filing condemnation suits have resulted in extensive and excessive delays. The recent acquisition of two fully analytical stereoplotters for photogrammetric surveying have increased the capabilities of the administration to meet time schedules. Furthermore, the addition of infra-red light and electronic distance measuring devices for field surveying has provided the means for accurately locating in the field the sites for towers which have been designed through the use of photogrammetrically surveyed centerline profiles.

Early attempts using Kelsh plotters to survey transmission line centerline profiles and stake tower centers in the field by survey crews equipped with theodolites and surveyors' chains were unsatisfactory because the crews were not able to precisely locate on the ground the locations which had been determined from the centerline profile. Accuracies of 1:5,000 in both photogrammetric and field surveys frequently resulted in differences of over 4 ft (1.2 m) within a

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Note.—Discussion open until April 1, 1982. To extend the closing date one month, a written request must be filed with the Manager of Technical and Professional Publications, ASCE. Manuscript was submitted for review for possible publication on April 22, 1980. This paper is part of the Journal of the Surveying and Mapping Division, Proceedings of the American Society of Civil Engineers, ©ASCE, Vol. 107, No. SU1, November, 1981. ISSN 0569-8073/81/0001-0011/$01.00.

segment of the transmission line survey 1 mile (1.5 km) long (a discrepancy of 0.8 m/km) between coordinated positions established photogrammetrically and those points as staked in the field.

In addition, the accuracy of the Kelsh plotter depended on the precision of the plotter operator's placement of a pencil in a chuck on a scaled point on a line drawn between two scaled coordinate positions. Plotting at a scale of 1:2,400, an error in positioning of only 0.04 in. (1 mm) would cause an error of 8 ft (2.4 m). If all errors introduced through the lack of precision of the plotter and the field survey crew were accumulative, in a 1 mile (1.6 km) tangent, the difference between the photogrammetric distance and the ground measured distance could be as much as 16 ft (3 m/km). This difference would be reflected in the location of tower centers on the ground. Also, in progressing along the centerline field, crews would differ progressively in the distance from the supposedly correct site. In any but extremely flat terrain, these differences would considerably change the relative elevations within each tower site survey.

Differences between the tower site surveys for design of the tower footings, and the ground encountered at the time of construction, inconvenience construction inspectors and can cause added expense for the contractors' erection of the towers. The tower footings are designed to withstand uplift forces by burial at a given depth, and the stub angle from the footing to the tower leg is a given length. Designed tower height is provided by a series of leg extensions between the footing stub angle and the tower body proper which are designed in increments of 2.5 ft (0.76 m). The bolt at the working point of the footing stub angle must further be at least flush with the surface of the ground and can extend no further than 2.5 ft (0.76 m) above it. If the ground is determined to be different in elevation from that surveyed for the design, it may be necessary to raise or lower the entire tower or raise or lower one or more working points to provide the required cover. Lowering the tower or one of the working points can result in extra digging, removal of material, and backfilling. Raising or lowering a working point also requires a change of the steel members to add or delete one leg extension unit. Since the material for each tower has been supplied on the basis of the design, it becomes necessary to investigate trading leg extension sections which can be modified between towers. In the event the required pieces are not available, it becomes necessary to change an entire tower or to order new material.

Differences in photogrammetrically determined elevations and those obtained in the field can result from a number of factors, but the two most frequently encountered are obscured ground and the previously noted difference in the physical location of the two surveys. Trees, brush, grass and debris from logging, or similar activities, prevent an accurate determination of the elevation of the ground because the plotter operator cannot see where the ground is. Obscured centerline elevations and tower sites must be obtained by field surveys. Obscured profile data and tower site surveys are provided for the design engineer only in cases of exigency when waiting for field data would delay the design critically. In such instances, the data is noted "approximate" and the engineer is aware of its deficiency.

The physical location of the surveyed sites can differ from the planned location if the accuracy of measurement is not adequate, as has been described. With both photogrammetric and field surveys at an accuracy of one part in 10,000,

the photogrammetrically determined positions can be located accurately on the ground. Bonneville Power Administration now has the capability for this accuracy in photogrammetric and field surveys.

This paper examines the development of photogrammetric and field procedures, the acquisition of equipment to provide adequate information for design, and specific applications of the fully analytical stereoplotters for the surveys of the 158 mile (254 km) Buckley-Summer Lake 500-kV transmission line and the 55 mile (89 km) 230-kV Okanogan Area Service Project. Other photogrammetric programs and techniques not related to these two projects have been developed but are not reviewed in this paper.

Development of Photogrammetric Capability

The Bonneville Power Administration had acquired a Kelsh plotter—an optical reconstruction stereoplotter—prior to 1963, but inadequate control surveying capabilities limited its use to localized mapping. This use was generally limited to surveying small tracts on which entry had not been granted, or to small substation areas, and without adequate control the surveys frequently did not meet the requirements of the design engineer. A further need was perceived to expand photogrammetric mapping to allow route comparisons during reconnaissance to aid in the selection of the final location. As a result, an additional Kelsh plotter and a cubic electrotape were acquired in 1964. The Kelsh plotters were used primarily for cross-sectioning for substation grading design, but experimentation was started on transmission line surveys. The Electrotape and a wild T-2 theodolite were used to establish all field survey and photogrammetry control.

The first photogrammetric survey of a transmission line was done in 1965 for a 230-kV transmission line between Spokane, Washington, and Boundary Dam, on the Canadian border. This line was parallel to an existing line, and the profile of the existing line was used to control the stereoplotter models, with tower positions serving as horizontal control points. In general, the profile and tower site surveys were adequate, but several problems appeared. A number of the structures of the first line were not constructed in the positions shown on the design profile, creating difficulties for stereoplotter operators in fitting the model to the ground. In one instance, a large boulder included in the tower site study was removed prior to tower erection, and a difference of over 5 ft (1.5 m) resulted between the design and construction elevations. It became apparent that a more accurate procedure of field editing and checking was needed to permit photogrammetric surveys to be fully usable. This required the establishment of coordinates on specific points which, along with a need for more accurate cross-sections for substation grading design, caused us to acquire electronic digitizers interfaced to a card punch to collect data on cards and allow computer drafting of the profiles and cross-sections.

A fully digitized Galileo-Santoni Stereocartograph V was purchased in 1967 to provide greater stereoplotting capabilities through the use of aerotriangulation to increase the density of control, and capability for greater precision and accuracy provided by the mechanical reconstruction stereoplotter. The digitizer was interfaced to a card punch and, like the Kelsh plotters, when being used for profiling, the data was punched on cards which could be processed through

a CDC 6500 computer to draw the profile on a flatbed plotter. This unit did provide a much higher degree of accuracy, but still required the operator to center the plotter on tower sites by aligning over a scaled position along a scaled pencil line. Problems still resulted from the minor inaccuracies in positions which caused some deviations from the surveyed positions. A greater problem resulted from the inability of the field crews to match the small error of closure (1:10,000) of the stereoplotter.

A second Galileo-Santoni stereoplotter, a Stereosimplex II, also fully digitized, was added to the photogrammetry equipment when the demand for photogrammetric surveys outgrew our capabilities. This unit also provided a relatively high degree of accuracy but has the drawback in profiling of the operator having to use scaled positions on a pencil line for setting over tower sites. One improvement in taking tower sites with this instrument was the pantograph movement of the plotter which permitted the use of a template to control the tower site surveys.

Programs written for processing data from the two Santoni plotters convert machine coordinates to ground coordinates, and the error in the alignment of the centerline profile survey and the error in position of each tower site survey is returned to the plotter operator. If errors are beyond a limit established to insure that design data does not differ from ground elevations by more than 6 in. (15 mm), the model is reset and the profile or tower sites are resurveyed.

The problems encountered in staking tower sites in the field—that is, that the field survey crews could not establish exact positions of coordinates on the ground—was resolved by equipping the crews with new infra-red light distance measuring devices. Survey crews are presently equipped with Hewlett-Packard models 3800 and 3810-A which provide an accuracy of 1:10,000 in linear measurement.

The Santoni mechanical reconstruction stereoplotters, coupled with the field surveys capabilities, established the ability to survey for design and insure the design would meet the construction standards. However, we were still not meeting the need which had been identified initially of providing profiles of alternate routes for reconnaissance analysis. The time required for setting the models in the stereoplotters, and for traversing along the proposed centerlines, limited production to an extent which precluded any substantial amount of route comparison profiling. It became apparent that full utilization of current developments in the fields of photogrammetry, remote sensing and digital terrain analysis to improve the administration's capabilities for environmental analysis, transmission line route selection, engineering design, and mapping required more versatile photogrammetric equipment.

Analytical Stereoplotting System

When the Bonneville Power Administration began acquisition of a fully analytical stereoplotter in 1975, no instrument meeting the photogrammetry needs was available. A number of functional requirements were determined based on achieving five goals: (1) Data gathering for environmental and reconnaissance analyses without entry on privately owned land; (2) increased production capabilities for surveys for design; (3) replacing the danger tree monitor (the danger tree monitor is a device for determining those trees which are dangerous

to a transmission line and must be removed. It consists of the two Kelsh plotters with their digitizers interfaced with a Honeywell 316 computer which has been adapted to a casette drive. Conductor configuration data, ground cross-sections, electrical clearance criteria, and tree growth rates are entered through the casettes. Tree top elevations are read by the plotter operator, and top-of-tree falling arcs are compared with the conductor location to determine dangerous trees. Trees into which the conductor could swing when blown by the wind are also identified); (4) meeting an increased work load without increases in manpower; and (5) development of new procedures for reconnaissance, surveying, and design. The proposal of Bendix Corporation to develop an analytical stereoplotter was accepted.

The Bendix Universal Stereoplotter, Model 1, (US-1) is a high-order stereophotogrammetric instrument, controlled by a 32K 1135 Digital Equipment Corporation computer. It can be described as an analytical reconstruction stereoplotter. In addition to the viewer, interface, computer and 52 in. × 42 in. (1,321 mm × 1,067 mm) flatbed data plotter, the configuration includes two 1.2 million word disk drives, a 7-channel magnetic tape unit, a 300-per-minute card reader, a hard-copy output device, a CRT unit, and a keyboard for communication with the computer. The viewer can accommodate all types of photography up to 9 in. × 9 in. (229 mm × 229 mm) format. The viewing optics can be zoomed manually from 5–30 diameters, which is an excellent feature used continually by the operators.

The general purpose software for the US-1 was supplied by Bendix, while programs for the centerline profile and tower site surveys have been written by Bonneville programmers. The general purpose programs used most frequently in transmission line surveying are those for aerotriangulation adjustment and model set-up. The system includes programming for aerotriangulation by monocomparator or stereo-comparator, or by dependent base-in, base-out procedures. Our preference is for the latter. The adjustment program for the aerotriangulation is dependably accurate. Because of the size of the computer, aerotriangulation is limited to strip adjustment.

For a computer-oriented individual with a good background in operation of a mechanical reconstruction stereoplotter, the analytical stereoplotter can become relatively easy to operate. As with a mechanical reconstruction instrument, diapositives are first inspected to identify control points. They are then PUG marked to provide a minimum of three pass points per model. Stereopairs are then placed on the viewer stages and the identification of each photograph is entered through the keyboard. The stages of the analytical stereoplotter differ considerably from those of an analog instrument. The viewing lenses are fixed in position and the stages move over them only in X and Y directions. Points on the photographs are read by the computer through the encoders which register stage coordinates. Following interior orientation, these are converted by the computer to photo coordinates, and these in turn to state plane grid system coordinates after absolute orientation. After the model is set up, the operator reads only state plane coordinates on the CRT display.

Processing a strip of photography on the US-1 begins with aerotriangulation. With the photographs in place on the viewer stages the fiducial marks are visited to enter the coordinates of each in the computer. Control points and pass points are then visited, identified, and entered. The computer, having been given the

state plane coordinates of the control points, adjusts the strip and advises the operator of the error of closure and residual errors in the adjustment. If the adjustment is satisfactory, the coordinates of all points are stored for future set-up of the models. Absolute orientation is accomplished for all additional set-ups by advising the computer which photograph is on each stage and visiting the fiducial marks of each. The computer recalls the stored data and re-establishes the model in precisely the same orientation as the initial set-up. This feature of the US-1 has reduced model set-up time from 1.5 hr to 15 min, a reduction to one-sixth of the time formerly required!

Transmission Line Surveying

Any photogrammetric project begins with planning the control net and covering the photography. Control monuments are set and prepaneled as closely in time to the taking of the photography as possible. Surveys for design are normally accomplished from photography taken with a 6 in. (152 mm) focal length camera from a flight altitude of 6,000 ft (1,830 m) above average terrain. Color film is used if the photography is to be used for danger tree identification or land use interpretation, but otherwise the film is black and white. Diapositives are made on glass plates or mylar film after inspection and acceptance of the photography.

Using the field control data and necessary additional PUG marks, the aerotriangulation of each strip is accomplished. When that is completed, manuscript bases are prepared on 3.5 ft by 6 ft (1.1 m × 1.8 m) mylar sheets. A coordinatograph is used to draw 5 in. (127 mm) grid lines representing 1,000 ft (305 m) intervals at a compilation scale of 1:2,400. The PUG marks and ground control points are plotted, annotated, and designed by elevation above sea level. The manuscript is taped to the flatbed data plotter and oriented for the computer by operator identification by X-Y coordinates of any four grid intersections, preferably near the corners of the area to be mapped. There is no limit to the skew of the manuscript from the orientation of the photographs the map can be 180° from the photographs.

The centerline for mapping and profiling is entered either by photo-identification or by the establishment of the coordinates of each angle point by keyboard entry or by plot on the manuscript. A planimetric strip map approximately 1,000 ft (305 m) to each side of the proposed line location is compiled and, after review by the reconnaissance engineer of the planimetry, the models are reset and the profile recorded. Profile data is collected on a disk, then transferred to a magnetic tape compatible with the CDC 6500 computer in which it is processed and plotted on profile paper by the flatbed plotter of the computer center. The map editor in the photogrammetry group traces a narrow strip of culture on the lower edge of the profile sheet to assist the sagger in adjusting tower positions, and to make him more aware of roads or utilities crossing the centerline which might require other than normal conductor clearance.

Upon completion of the sagging and determination of the tower centers, data cards are prepared containing the station, tower designation, and elevation of the tower center as scaled from the profile. Additional cards are prepared for angle towers containing the state plane coordinates and centerline station. When the models are again reset, the stereoplotter, upon command through the keyboard,

travels to the center of a tower and places the index dot at the elevation scaled from the profile. The operator corrects the elevation as necessary and records it by depressing a foot pedal which also directs the stereoplotter to move to the first of a series of readings along the diagonal on which the center of a leg will fall. The operator profiles that leg, then commands the system to move to the first point on the next leg to be profiled. After completing the survey of the tower site, the operator can terminate the model or move to the next tower along the line. This data is processed through the CDC 6500 to prepare a list of required steel members to be ordered.

A copy of the profile and tower site listings are given to the field survey crews to stake the towers in the field. Using previously set control points, the angle points are set by coordinate position, and intermediate towers are set by projection of the tangent or by coordinates if in an inaccessible location. Elevation differences between adjacent towers are measured with the infra-red light instruments and compared to the photogrammetrically determined differences. A discrepancy of more than one foot is cause for rerunning the profile in the stereoplotter. After determining that no soils, land use or other problems could cause the tower to be moved, field crews place monuments at each tower site for permanent reference.

Okanogan Area Service 230-kV Project

Increased electrical loads in the Okanogan Valley in north central Washington State required new transmission facilities from Chief Joseph Dam on the Columbia River for the cities of Okanogan and Omak and the town of Tonasket, a total straight-line distance of about 54 mile (87 km). The initially planned route crossed the Colville Indian Reservation the full distance from the dam to Okanogan, and approval was not given by its tribal council for that route. As a result, another route by way of the town of Brewster, about 12 mile northwest of the dam, was considered the only available alternative, and control surveys were completed on that route. The original route was still preferred, and negotiations with the tribal council for approval to cross the reservation continued.

With the time for design and ordering materials for construction of the line becoming critical, it became apparent that it would be necessary to survey the centerline profile and the tower sites photogrammetrically. The reconnaissance and location of the final centerline was completed for only small segments of the project when the photogrammetry and remote sensing unit was directed to proceed with the surveys. Control points, and those section corners which had been found during the control survey, were targeted with panels 1 ft (0.3 m) wide and 6 ft (1.8 m) long to form a cross 12 ft long overall. Photography at a scale of 1:12,000 was obtained and aerotriangulation and profiling begun. During this same period, a contract for completion of the surveys in the field was negotiated on the understanding that the contractor would use photogrammetrically determined coordinates to establish the centerline. Other work of the contract was to monument tower centers, check the differences in the elevations between tower centers, locate tie section and individual property corners, and survey roads for access to the towers.

Negotiations with the Colville Business Council suddenly were concluded

with permission granted to survey the transmission line across the reservation. The plan of service was changed, and all surveys completed between Chief Joseph Dam and Omak were voided. Since time had become more critical, both the US-1 and the Stereosimplex II were used for the photogrammetric surveying. Time constraints became more of a problem for the operators because numerous changes in the proposed alignment were made throughout the time they were working. Furthermore, the contractor was asking for data needed to complete projection of the centerline to determine the extent of recovery of section corners necessary for adequate mapping and right-of-way description preparation, as well as to complete access road surveys before winter snows obscured existing trails.

Surveys over 35 mile (56 km) of the route were accomplished with the US-1, and surveys over 30 mile (48 km) were done with the Stereosimplex, including 9 mile (145 km) of revisions. These required 51 man-days of operator time on the US-1 and 74 man-days of operator time on the Stereosimplex. The most significant differences were in time requirements for aerotriangulation and tower site surveys. The following tabulation compares the time taken for each task performed in the surveying with the US-1 and Stereosimplex, and that for similar work done by field survey crews in similar terrain and vegetation.

TABLE 1.—Man-Hour Comparison of Various Surveying Methods

Task (1)	Unit (2)	Man-Hours per Unit: US-1 (3)	Stereosimplex (4)	Field crew (5)
Aerotriangulation	Mile	1.4	4.3	—
Planimetric mapping	Mile	4.6	4.3	—
Profiling	Mile	3.4	5.3	24.0
Tower site surveys	Each	0.5	1.1	6.4
Total survey	Mile	11.7	19.7	105.3

The field survey crew hours are not available for any work comparable to aerotriangulation, which in field terms would amount to preliminary location line, wiggling or triangulating for projection of the centerline, and similar requirements. Planimetric mapping would include not only the time spent in drawing a hardshell map in a field office, but such work as obtaining topography in the field. The field survey crew total survey hourly requirement per mile is obtained by subtracting the hours spent by the field contractor on the Okanogan project to perform his work from the average hours required to do a complete project under normal field conditions in similar terrain.

The quality of the photogrammetric work was fully acceptable. In fact, improper field procedures by the contractor in making adjacent tower elevation checks were discovered by his disagreement with the photogrammetrically determined elevations. Upon correcting those procedures, he was in agreement. The field work was done with electronic distance measuring devices, and no errors in photogrammetric distances were discovered.

Because of the many changes in the plan of service and in the alignment, considerable additional costs were incurred in control surveying, and the comparison of this cost to an average project is not valid. For that reason, we have not tried to present a dollar cost or savings figure for the total project.

Buckley-Summer Lake 500-kV Transmission Line

The Buckley-Summer Lake 500-kV transmission line extends from the intersection of two major transmission line corridors north of Maupin, Oregon, parallel to the PacificPNorthwest-Pacific Southwest AC Intertie line to a new substation site to be constructed near Silver Lake, Oregon at the junction of the Intertie and a 500-kV line to be constructed by the Pacific Power and Light Company. Aerotriangulation, centerline profiling, and tower site surveys of this 158 mile (254 km) line were done entirely with the US-1 analytical stereoplotter.

The centerline of the Buckley-Summer Lake is approximately 100 ft (30 m) from the centerline of the intertie, which makes a considerable difference in the field preparation for the survey from that required on the Okanogan project. During the survey for the intertie, monuments on which Oregon state plane coordinate system coordinates were established were placed at each tower center. Control for the photogrammetric survey was established by placing a target on every fifth tower, at a spacing of approximately 1 mile (1.6 km). The elevations of the monuments had also been established, and this data was supplemented by the profile of the original survey. Wing points were established about 1,000 ft (305 m) from the centerline on positions which were photo-identifiable at a 2–5 mile (3–8 km) spacing to prevent rotation about the longitudinal axis to a degree which would affect the usefulness of the survey. Since land ownership had been determined during the original survey, no targets were placed on section corners or other property boundaries.

The control survey of the intertie line had been run as a traverse between USC and GS positions with a closure of 1:10,000, and was not completely adequate to control the stereomodels. In a number of instances, poor adjustments caused resetting of a long strip of photography which increased the time required for aerotriangulation. Even with this problem, the time for aerotriangulation was only 56 man-days for the 158 mile of transmission line.

Man-hour requirements to date for this project have been as follows: for aerotriangulation, 3.0 man-hr/mile for the combination of planimetric mapping and profiling the centerline, 3.2 man-hr/mile and for the tower site surveys, 0.3 man-hr/site. The combination of mapping planimetry, profiling centerline, and surveying tower sites for the Buckley-Summer Lake line was 7.4 man-hr/mile as compared with the 11.7 man-hr/mile achieved on the Okanogan Area Service Project. The terrain traversed by the two projects is quite similar, but the Buckley-Summer Lake line passes through scattered juniper trees through about two-thirds of its length and through pine trees in two sections totalling 21 mile (34 km) in length. The juniper trees will not be cleared, and to insure that the conductor height is sufficient to provide the required electrical clearance, a random sampling of the tree heights was made. In very few instances were the trees dense enough to cause difficulty in reading the ground elevations for the profile and tower sites.

Conclusions

The experience of the Bonneville Power Administration has been that for specialized surveying requirements such as transmission line design and cross-sectioning for grading of foundation areas for structures such as substations, photogrammetric surveying can achieve tremendous cost savings. In addition, the use of a fully analytical stereoplotter can add to those cost savings in an amount which can pay for the purchase of the equipment in a relatively short time if it is regularly utilized.

Surveys conducted by operators using both mechanical reconstruction stereoplotters and fully analytical stereoplotters are fully adequate for the design of a transmission line in any areas where the plotters can be accurate, that is, in open terrain relatively free of dense vegetation.

The cost savings of the analytical stereoplotter result primarily from types of surveying which require frequent resetting of previously oriented models. For topographic or planimetric mapping in which a model is set up and used for mapping for a relatively long time, any differences in speed of operation appears to be the result of differences in the speeds of the operators in doing the mapping itself. The use of the analytical stereoplotter is recommended whenever the capability of the on-line computer can be of advantage.

STANDARD FOR SYMBOLOGY ON ENGINEERING SCALE MAPS[a]

By Robert P. Jacober, Jr.[1]

INTRODUCTION

At the present time, a set of map symbols, universally recognized as a national standard for large scale maps, does not exist. Though many professional, governmental, and commercial organizations have symbols used internally, little has been done to consolidate the various lists into one standard legend.

In 1978, Dean Merchant, the Chairman of the Cartographic Surveying Committee of the Surveying and Mapping Division, asked the author of this article to develop an appendix for the forthcoming ASCE manual, *Manual on Map Uses, Scales, and Accuracies for Engineering and Associated Purposes*. The appendix would contain a legend of nationally accepted symbology for use by ASCE members as standard symbols on large scale maps (1:240–1:4800). It was found that many organizations have been studying the problem or emphasizing the need for such a standard, but no actual standard existed. The development of a set of symbols that could be acceptable as the nucleus for a national standard became the subject for the author's Master's thesis (2) and for Chapter V of the ASCE manual entitled, *Map Content and Symbology*. Since Chapter V will be published for review and comment in the *Journal of the Surveying and Mapping Division*, this paper will present the background to the chapter and describe the problems encountered in the development of the symbol system.

The need for standard symbology was stated as far back as 1938 by the National Resources Committee (3). In 1948, Walter Blucher, executive director of the American Society of Planning Officials made the following statement: "Here in the United States it is almost impossible to compare drawings prepared by different draftsmen or offices, not only because they may be of different scales, but because the symbols used are often as far apart as the poles (5)." Joe Steakley reiterated this need in a letter to the American Cartographer in 1977 (4). And today, the need for a standardized set of symbols is more important

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Note.—Discussion open until April 1, 1982. To extend the closing date one month, a written request must be filed with the Manager of Technical and Professional Publications, ASCE. Manuscript was submitted for review for possible publication on April 24, 1980. This paper is part of the Journal of the Surveying and Mapping Division, Proceedings of the American Society of Civil Engineers, ©ASCE, Vol. 107, No. SU1, November, 1981. ISSN 0569-8073/81/0001-0021/$01.00.

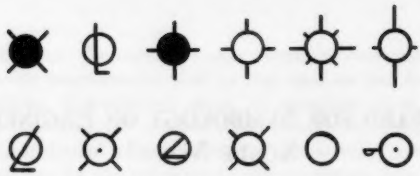

FIG. 1.—Variety of Symbols Representing Free-Standing Light Poles

with computer assisted mapping becoming the norm rather than the exception.

One example of the lack of standardization is the features that are represented by a circle: manhole, light pole, chimney, oil tank, airport taxiway light, proposed location for a tree, utility pole, oil or gas well, sump, and mill. Confusion could result if one organization which used the circle to represent a manhole requested another organization's manuscript that used the circle to represent a light pole. If the manuscript did not contain a legend, the information represented would be meaningless to the requesting organization. A second example of the lack of standardization is demonstrated by the symbols used by various organizations to represent free-standing light poles (Fig. 1).

Developing a Standard Symbology

The problems in developing a standard symbology are numerous:

1. A unique symbol for each feature to be represented must be created.
2. The symbols must be easily programmable for computer.
3. A method to differentiate between proposed, existing, and intermittent, destroyed, or abandoned features has to be included in the system.
4. A procedure must be established to phase in the symbols.
5. A procedure has to be initiated that will maintain the currency of the symbology file by adding symbols as new features need to be represented.

The crux of the entire issue is stated in the first problem, to insure that only one symbol exists for each feature. To solve the first problem, a folio of legends was assembled. This file included symbol lists from large and small private companies in the U.S. and abroad; national and international professional organizations; city, county, state, regional, national, international, and foreign governmental agencies; and military and educational institutions, both domestic and foreign. All features represented in the legends were listed. Next to each feature name were drawn all of the symbols used to represent that feature. The criteria used to select a unique symbol to represent each feature were popularity and ease of computer programming, described as follows:

1. Popularity is an objective criterion. If a symbol is almost universally recognized as representing a feature, i.e., an X for a benchmark, that symbol-feature relationship should be retained.
2. The easily computer programmable criterion is subjective in that it is the

author's conception of what is easily programmable in Fortran for use on the Versatec electrostatic plotter.

For some features, the symbol selection is easy. For example, a horizontal control station, which is internationally represented as a triangle is also easily programmed. That symbol easily met the criteria. For other features, such as a free standing light pole, the choice is more difficult. The final list of features and the symbols that represent them is incorporated in Chapter V.

The selection of symbols based on how easily one may program them for computers is directed toward the increased use of computer driven plotters and Cathode Ray Tube (CRT) devices. The author programmed each symbol listed in Chapter V to insure that they could be displayed using the computer. An additional advantage to having unique symbols for each feature is that computer assisted map reading and reproduction becomes easier. Instead of storing a separate symbol software package for each map produced by a different organization, only one symbol software package need be stored.

The symbols are designed to be used in a monocolor production process, i.e., black or blue on white or clear, clear on red or black, etc. This does not preclude the use of color to help differentiate between classes of features. For example, on the same manuscript, use black to represent roads, red to show power distribution, blue for water distribution, etc. With the aid of computers and memory files, the overlay system could also be used in a monochrome or multicolor display. Each feature layer is printed on a separate sheet of transparent or translucent material. With the monochrome system, each layer would be printed in the same color. With the multicolor system, each feature class could be printed in a different color on the separates. The overlay method could be used to help solve the third problem, how to represent existing, proposed, and destroyed, abandoned, or intermittent features.

Depiction of existing features and proposed features on the same manuscript is tied to the purpose of that manuscript. And the purpose of the map dictates what features belong on the map and how they will be portrayed. For example, the maps produced by a state highway department will probably depict the roads as parallel lines. If the map is used by the planning division, the *proposed* roads will usually appear as solid lines and the existing roads as broken lines. For the highway maintenance division, the *existing* roads will appear as solid lines while the proposed roads will appear as dashed lines. Though the two divisions work for the same agency, if a person from the maintenance division saw an unlabelled map produced for the planning division, he would probably interpret the map erroneously. This problem can be greatly reduced or eliminated by use of appropriate titles, legends, and overlays on all maps.

Conclusion

Once the standard symbol system is accepted, the question of how to implement the system must be addressed. With many years' accumulation of irreplaceable manuscripts in files across the United States, recompiling maps with a standard symbol system would be unnecessary and impossible. As long as legends are available for the filed maps, they will remain valuable documents. But as new maps are produced or old maps revised, the standard symbols should be used,

especially on maps that are being digitized. Thus over a period of years, all maps will be produced using the standard symbols.

A major problem with the proposed list of symbols contained in Chapter V, is that it is not comprehensive. The features represented are those most often used on the engineering scale maps that the author had accessible to him. Special use symbology, infrequently used symbols, or symbols not on the maps used by the author were not included in the chapter.

As new symbols need to be added to represent new features that advancing technology develops or to represent features that were not included in the original list, a method must be available to update the list. As a map producer designs a new symbol, that symbol should be sent to the Committee on Cartographic Surveying, Surveying and Mapping Division, ASCE, for inclusion in the symbol list. The symbol should also be sent to the Committee for publication in the *Journal of Surveying and Mapping Division* for comment and review.

Though Chapter V is aimed at solving the problem of symbol standardization for large scale maps, the chapter also recommends cartographic guidelines that should be used by all within ASCE who produce maps or plots. The *Manual on Map Uses, Scales, and Accuracies for Engineering and Associated Purposes* is being published to provide information to enable the map designer or user to select the proper type, scale, accuracy, and quality maps suitable for the map's intended purpose, which will promote standardization within the American Society of Civil Engineers (1).

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The Surveying Engineer and NAD-83[a]

By J. E. Colcord,[1] M. ASCE

Introduction

Historically, geodetic data and the choice of projections began with Eratosthenes (about 200 BC) with his measurements between Alexandria and Syene (3) and in his production of a map of the Mediterranean area. In geodesy two data are usually considered: horizontal ($\Phi_0, \lambda_0, \tau_0, a, f$) and vertical (Mean Sea Level, MSL, and possible geoidal height, $\mathcal{N}$). The work of Delambre and Mérchain (7) in 1792 measuring the Dunkirk-Barcelona arc resulted in the determination of the meter (the ten-millionth part of the terrestrial meridian and fixed at 3 pieds, 11.296 lignes (old measure)) by the Institute to the Corps Législatif on June 22, 1799. In the U.S., the horizontal data have been: New England Datum of 1839; U.S. Standard Datum of 1901; North American Datum (NAD) of 1913; and NAD-27—all using the Clarke Spheroid of 1866. NAD-27 had Meades Ranch as the initial point with the initial azimuth to Waldo. For this data major loop closures of approx 1 part in 100,000,000 over continental distances were obtained, but locally some work was less precise. Since then, there have been vast improvements in measuring and adjustment techniques and extensive re-observation and re-adjustment. This means that the net is not now satisfactory for the user community, and, thus, a new adjustment was begun in 1974 (1), resulting in the North American Datum of 1983—NAD-83.

NAD-83

The National Geodetic Survey, NGS, along with representatives of Canada and Mexico, made arrangements in 1973 to carry out a cooperative adjustment. Since then, representatives of Denmark and Central America have joined the cooperative program. The formidable tasks include: (1) Placing all existing data in computer readable form; (2) analyzing the data to recommend additional observation; (3) establishing a multipurpose data bank; (4) performing research and development in adjustment methods; and (5) choosing a new datum. The

[a]Presented at the April 14–18, 1980, ASCE Convention and Exposition, held at Portland, Oreg. (Preprint 80–073).

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immensity of the task is apparent since more than 200,000 points and more than 3,000,000 observations are considered. Since the original adjustments for NAD-27, later work has been forced to fit "fixed" control. In the case of NAD-83, the effect of a new rational adjustment affects all points and, thus, all users and products based on geodetic control whether it be maps or surveys in geodetic coordinates or plane coordinate systems. Serious problems involve

```
                                   ***PRELIMINARY***                                  PAGE--022
US DEPARTMENT OF COMMERCE - NOAA    HORIZONTAL CONTROL DATA              QUAD--L38082243 QSN--0001
NOS - NATIONAL GEODETIC SURVEY      NORTH AMERICAN DATUM 1927            STATE--WV  DIAGRAM--NJ 17-4
ROCKVILLE MD 20852 - OCT 1977      ADJUSTED BY--CGS    YEAR--1958        COUNTY--WAYNE
                                         SOURCE--G11678

STATION NAME--NEWCOME                                      PRIMARY        GEODETIC AZIMUTH (FM SOUTH)
     TYPE--2ND-ORDER TRIANGULATION                         AZIMUTH REFERENCE OBJECT--    (DEG MIN SEC)
   EST BY--CGS    YEAR--1957                               NEWCOME AZ MK                  24 34 54.0

                                  UNIVERSAL TRANSVERSE MERCATOR   POINT    CONVERGENCE    GRID AZ TO
     GEODETIC POSITION               COORDINATES IN METERS        SCALE    OF MERIDIAN  AZ REF OBJECT
      (DEG MIN SEC)             ZONE  X-EASTING  Y-NORTHING       FACTOR  (DEG MIN SEC) (DEG MIN SEC)
  LAT  38 18 31.23770N           17  369099.847 4240918.191    0.99981102   -0 55 41.5    205 30 35.5*
  LON  82 29 49.75970W           16  893779.358 4249460.968    1.00151017   +2 47 41.6    201 47 12.3*

  ELEVATION (MTRS)  297.1   SOURCE OF ELEVATION DATA--TRIGONOMETRIC LEVELING   MODELED GEOID HEIGHT (MTRS)  -2.5

                    STATE PLANE COORDINATES IN SURVEY FEET AND METERS   PT SCALE   CONVERGENCE    GRID AZ TO
        STATE-ZONE CODE    X (FEET)     Y (FEET)     X (MTRS)     Y (MTRS)    FACTOR    OF MERIDIAN  AZ REF OBJECT
            WV-S   4702   1570427.60   479995.27   478667.291  146302.850  0.99992802   -0 55 31.9    25 30 35.9*
            KY-N   1601   2502946.72   299268.69   762899.606   91217.279  0.99996589   +1 05 25.3    23 29 28.6*

                                                             *CAUTION - ARC-TO-CHORD CORRECTION ASSUMED ZERO

                ****************** S T A T I O N   D E S C R I P T I O N ******************
STATION NAME--NEWCOME                              STATE--WV  COUNTY--WAYNE            QUAD--N38082243 QSN--0001
********************************** YEAR COP ****************************** REACHED BY ****** PACK TIME ** HGT OF TELESCOPE
MONUMENT BY--CGS                   1957 FAR   MKR TYPE--TRIANG STA DISK   TRUCK              00 HRS 00 MIN       15 METERS

          CODE MARK TYPE ********* SETTING/LANDMARK TYPE ************************************ MAGNETIC PROPERTY
    SURFACE--D09  SURVEY DISK       SET INTO THE TOP OF A SQUARE CONCRETE MONUMENT               UNKNOWN
UNDERGROUND--D04  SURVEY DISK       SET INTO THE TOP OF AN IRREGULAR MASS OF CONCRETE            UNKNOWN

              REFERENCE OBJECT ********** HEADING/DISTANCE (MEASURED OR ESTIMATED) ** DIRECTION
              DAVIS 2                                                                 0 00 00.0
         D09  NEWCOME AZ MK                   SSW  ESTIM APPROX 0.4 MI              135 03 51.3
         D09  NEWCOME RM 2                    WNW         42.89 FEET  13.073 MTRS   214 01 55
         D09  NEWCOME RM 1                    ENE         66.12 FEET  20.153 MTRS   358 50 15

STATION IS APPROXIMATELY 6.5 MILES NORTHWEST OF WAYNE, 3.0 MILES WEST OF LAVALETTE, NEAR THE HEAD OF NEWCOME BRANCH AND
ON THE HIGHEST POINT OF A HILL OWNED BY MRS. MARTHA WORKMAN.

TO REACH FROM THE JUNCTION OF U.S. HIGHWAY 60 AND U.S. HIGHWAY 52 IN HUNTINGTON, GO SOUTH ON U.S. HIGHWAY 52 FOR 6.3
MILES TO THE JUNCTION OF STATE HIGHWAY 75, KEEP LEFT ON U.S. HIGHWAY 52 FOR 0.85 MILE TO A SIDE ROAD ON THE RIGHT AND
SIGN LAVALETTE NURSERY 100 YARDS, TURN RIGHT, CROSS STEEL BRIDGE, KEEP LEFT AND FOLLOW THE MAIN TRAVELED ROAD FOR 2.85
MILES TO A SIDE ROAD ON THE RIGHT, KEEP LEFT, PASSING SCHOOL AND CHURCH ON THE LEFT AND GO 0.25 MILE TO A SIDE ROAD ON
THE RIGHT AND A HOUSE ON THE RIGHT, TURN RIGHT, CROSS STREAM AND FOLLOW ROAD ALONG STREAM FOR 1.5 MILES TO TOP OF HILL
AND A DIM ROAD ON THE LEFT, KEEP RIGHT AND FOLLOW ROAD ALONG RIDGE FOR 0.1 MILE TO THE AZIMUTH MARK ON THE LEFT.
CONTINUE ON ROAD FOR 0.3 MILE TO A ROAD ON THE LEFT LEADING TO MRS. WORKMAN'S HOUSE, TURN LEFT AND GO 0.2 MILE TO MRS.
WORKMAN'S HOUSE, PASS TO THE NORTH OF HOUSE ON FIELD ROAD AND GO EASTERLY ON RIDGE FOR 0.15 MILE TO THE HIGHEST POINT OF
HILL AND THE STATION.

STATION MARKS ARE STANDARD DISKS STAMPED---NEWCOME 1957---.  THE SURFACE DISK IS SET IN A SQUARE CONCRETE POST WHICH
PROJECTS 4 INCHES.  IT IS 12 FEET NORTH OF A PINE TREE WITH A TRIANGLE BLAZE ON THE NORTH SIDE.  THE UNDERGROUND DISK IS
SET IN AN IRREGULAR MASS OF CONCRETE 24 INCHES BELOW THE SURFACE OF THE GROUND.

REFERENCE MARK NO. 1, A STANDARD DISK STAMPED---NEWCOME NO 1 1957---, IS SET IN A SQUARE CONCRETE POST WHICH PROJECTS 4
INCHES.  IT IS 7 FEET SOUTH-SOUTHWEST OF A PINE TREE WITH A TRIANGLE BLAZE ON THE SOUTH SIDE, APPROXIMATELY 7 FEET
LOWER IN ELEVATION THAN THE STATION AND ON THE NORTHEAST SLOPE OF THE HILL.

REFERENCE MARK NO. 2, A STANDARD DISK STAMPED---NEWCOME NO 2 1957---, IS SET IN A SQUARE CONCRETE POST WHICH PROJECTS 4
INCHES.  IT IS 4 FEET EAST-NORTHEAST OF A PINE TREE WITH A TRIANGLE BLAZE ON THE EAST SIDE, APPROXIMATELY 6 FEET LOWER
IN ELEVATION THAN THE STATION AND ON THE WEST SLOPE OF THE HILL.

AZIMUTH MARK, A STANDARD DISK STAMPED---NEWCOME 1957---, IS SET IN A SQUARE CONCRETE POST WHICH PROJECTS 4 INCHES.  IT
IS 175 FEET SOUTHWEST OF A GAS LINE CROSSING, 125 FEET NORTHEAST OF THE NORTHEAST CORNER OF MR. OTIS CRABTREES HOUSE, 13
FEET NORTHWEST OF THE CENTER OF A DIRT ROAD AND 1 FOOT NORTHWEST OF A FENCE LINE.

HEIGHT OF LIGHT ABOVE STATION MARK 18 METERS.

                ************************ R E C O V E R Y   N O T E ************************
STATION NAME--NEWCOME                              STATE--WV  COUNTY--WAYNE            QUAD--N38082243 QSN--0001
********************************** YEAR COP ****************************** REACHED BY ****** PACK TIME ** HGT OF TELESCOPE
RECOVERY BY--CGS                   1961 JJC  CONDITION--GOOD               LIGHT TRUCK        00 HRS 00 MIN           METERS

          CODE MARK TYPE ********* SETTING/LANDMARK TYPE ************************************ MAGNETIC PROPERTY
    SURFACE--D09  SURVEY DISK       SET INTO THE TOP OF A SQUARE CONCRETE MONUMENT               UNKNOWN

THE STATION AND REFERENCE MARKS WERE RECOVERED AS DESCRIBED AND FOUND TO BE IN GOOD CONDITION.
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FIG. 1.—Preliminary Data Format (Ref. 8)

analysis of the reliability of the old data, rejection of outliers, and the reduction of this data to an assumed computational ellipsoid, including inputs of geoidal height and deflection of the vertical. The International Association of Geodesy has, at its Canberra, Australia, meeting recommended ellipsoid parameters as $a = 6{,}378{,}137$ m (1 m = 3,937/1,200 United States Survey Foot, USSF; and $f = 1/298.257$. The new datum is geocentric, and, as mentioned, geoidal height reduction is needed. There is no initial point, such as Meades Ranch, as the

datum refers to the entire network based on many points determined by satellite (Doppler) and other modern (Transcontinental traverse) methods. These stations, with appropriate weights, are then introduced into the adjustment, solving the more than 500,000 unknowns, simultaneously, by partitioning the equations into Helmert blocks of about 1,000 stations.

(a)

(b)

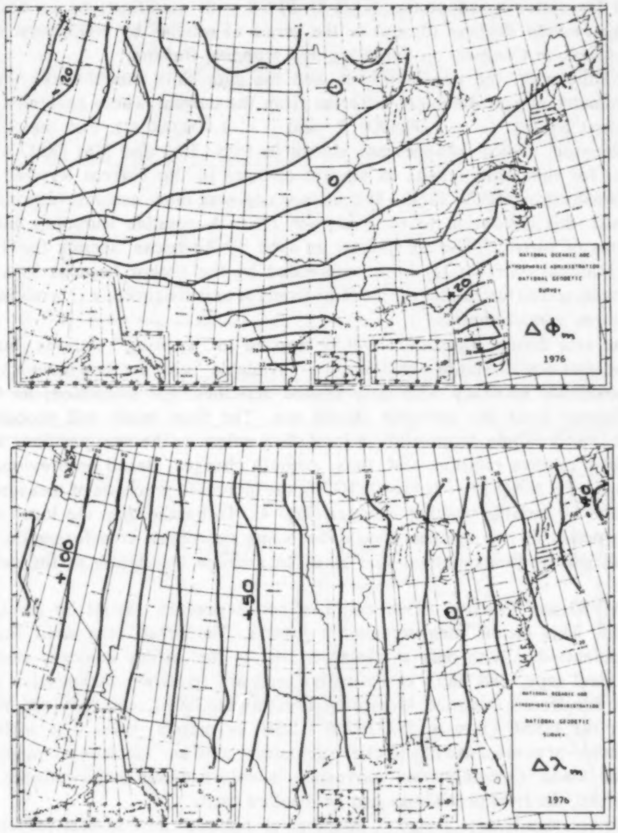

FIG. 2.—Expected Datum Change, in meters: (*a*) Latitude change; (*b*) Longitude change (Ref. 3)

NAD-83 and Plane Coordinate Systems.—To most survey engineers, the effect of NAD-83 on the State Plane Coordinate System (SPCS) is of special importance. To some, the Universal Transverse Mercator (UTM) with 6° zones, since it has one basic formulation, may be of interest. However, the scale factor and

the second term, θ, are significant even for T-I or transit work. In the preliminary data format as developed by NGS (Fig. 1) all values (SPCS, UTM, etc.) are available. This data is then followed by a description and survey notes.

The major significance of NAD-83 is (1) Use of Standard International (SI) coordinates as fundamental units; (2) the anticipated datum shift (Fig. 2) and rotation; and (3) the adoption of azimuth from north. The net result is the need for states to adopt the NAD-83 SPCS with modifications as described in the *Federal Register* (6) and in the series of articles by NGS personnel in the *American Congress on Surveying and Mapping Bulletin*.

For the user, the use of meters and the data shift should make the new coordinates appear distinctly different from the current where generally in the Lambert System ($x > 1{,}000{,}000$ ft, and $y < 1{,}000{,}000$ ft). For the State of Washington, typical changes are shown in Figs. 3(*a*) and 3(*b*) (Ref. 4), and must, for this state results in several changes in the current Revised Code for Washington, RCW:58:20. The survey engineer must assume responsibility to push this legislation to be ready for 1983. A possible stumbling block is the use of *meters*. (The SI spelling as used by American Society for Testing and Material (ASTM) (2) is recommended by the author because it is more universal and differentiates the machine from the unit distance, i.e., "a micrometer measures in micrometres.")

The new datum adjustment will be applied to "existing" stations, but it is hoped that not all stations that may be "available" now as they have a variety of positional accuracy with only limited statement (or admission) as to the confidence level the surveyor should use. The final result will probably be more than a simple datum shift in local Φ, λ unless as the new positions should result in relative rotation, and, thus, azimuth changes. Based on previous data changes, i.e. NAD-27 → NAD-27/KC76, the increased precision of measurement resulted in variable position changes (Fig. 4). To handle this, the local survey using points in two systems should use some coordinate transformation, such as the gravity point method that gives an average translation to the centroid (5).

NAD-83 and Practice.—One major effect in practice should be the use of the SI. This means measurement in meters. The result is a need for new equipment—but with only a minimum cost to the survey engineer since the rod, steel tape, and cloth or fiberglass tape are relatively inexpensive items. Most Electronic Distance Measuring Equipment, EDM, can now measure in either the USSF (1 m = 3937/1200 USSF) or meters; thus, it is a flick of a switch—or at most use of a pocket calculator—until an "electronics transplant" can be made. Certainly, this conversion is only a minor inconvenience, and, therefore, the field procedures can be handled easily.

A more significant change is updating of records, now in feet (or chains), and possibly "local" data. This conversion may be less straightforward, but can be done "as needed," when a modern survey ties to old data. Any old survey that was consistently done can be changed by a coordinate transformation equation to fit a consistent new boundary. Obviously, this is a typical computer job and can be done easily for small areas. For county or city records if a modern multipurpose cadastre, such as Land Registration Information Systems, LRIS; Modernization of Land Data Systems, MOLDS; and Computer Assisted Mapping and Records Activity Systems, CAMRAS has been adopted and the

multipurpose cadastre has been tied to a geodetic framework (9) and an orthophoto base map; then the advantages accrue and the conversion is done readily.

(a)

(b)

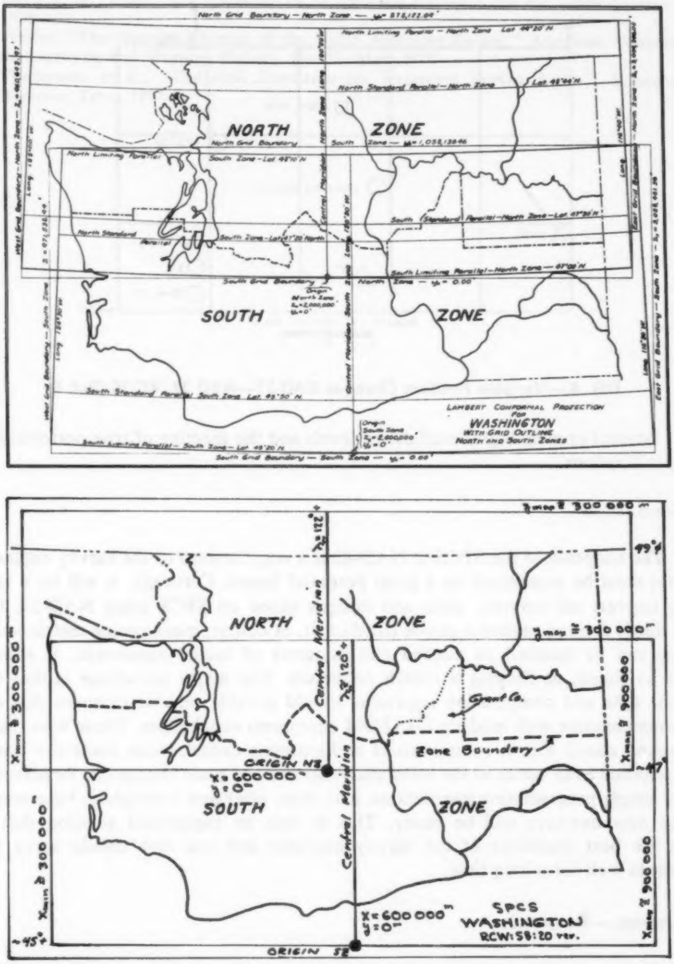

FIG. 3(*a*).—Current SPCS (RCW:58:20) for the State of Washington (Ref. 4); (*b*) Suggested New SPCS for Washington

A great advantage will be a knowledge of the consistency of the new NAD-83 net resulting in the real ability of a local survey engineer to tie to the SPCS

and obtain reasonable closures consistent with his modern equipment and procedure ability, thus, allowing least squares adjustment and the reporting

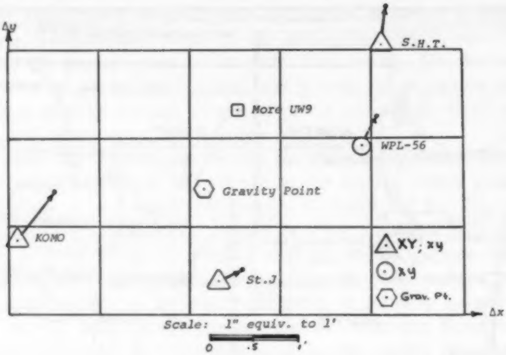

FIG. 4.—Variable Position Changes NAD-27—NAD 27/KC-76 (Ref. 5)

of rational or meaningful confidence levels and the meeting of true performance specifications.

Conclusions

The adoption of the SPCS in NAD-83 is a step forward to the survey engineer that must be recognized as a great potential bonus. Certainly, it will be a task to convert old surveys, plats and designs based on SPCS using NAD-27, but if this is done as needed it can be handled. It, of course, means going metric—this too can be handled as needed and in terms of major equipment. It should be as simple as flicking a switch to meters. The major advantage is that the new data and computation equations should provide reliable closures that are commensurate with modern T-2/EDM equipment capabilities. Those who really looked ahead with a computerized multipurpose cadastre can make the transformation even easier as the translation rotation and scale change can be affected by simple programming instructions, and, thus, after new orthophoto base maps, the new overlays will be ready. This is then an engineered solution that is in the best traditions of the survey engineer and one that should serve his clients well for a long time.

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Survey Control for I-205 Columbia River Bridge[a]

By John D. Howard[1]

Introduction

Traditionally, bridge layout for stream crossings has been accomplished by triangulating from points on a river bank base line. In 1962, a survey for the 21,697 Columbia River Bridge at Astoria was undertaken. A first order survey was desired and the easiest method looked like traversing using electronic distance measuring equipment. The EDM was tried, returned to the factory, tried again, but was not able to give consistent results so the layout method had to be changed to a second order triangulation. The best part of a year and a great deal of effort went into the triangulation. Apparently the results were acceptable as the bridge was built without any surveying problems which is the final proof of any method. Fifteen years later and a second crossing of the Columbia River; it was again our intention to do the layout by traversing. In this case the distances were shorter and the improvements in the electronic distance measuring equipment made its use desirable.

Site (Figs. 1, 2, and 3)

The Columbia River crossing of the I-205 freeway is a major link in the eastside bypass of Portland and Vancouver. Proceeding south from the Washington shore, twin structures each 7,460 ft (2,274 m) long cross the main north channel and join a 1,170 ft (357 m) section of fill on Government Island. From the south end of this fill, twin structures each 3,120 ft (951 m) long cross the south channel to the Oregon shore, giving the project an overall length of 11,750 ft (3,581 m).

The Oregon portion of the North Channel structure begins on the Washington shore with Span 10. Pier 11 is on dry land. Piers 12–26, which are in the river, plus abutment 27, complete this structure to Government Island. The spans are designed for either cast-in-place or pre-cast concrete segmental construction and have lengths varying from 600 ft (183 m)–242 ft (74 m). The layout covered

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Note.—Discussion open until April 1, 1982. To extend the closing date one month, a written request must be filed with the Manager of Technical and Professional Publications, ASCE. Manuscript was submitted for review for possible publication on April 24, 1980. This paper is part of the Journal of the Surveying and Mapping Division, Proceedings of the American Society of Civil Engineers, ©ASCE, Vol. 107, No. SU1, November, 1981. ISSN 0569-8073/81/0001-0033/$01.00.

in this paper will be confined to the North Channel and does not include the Washington approach or the South Channel structure.

The bridge alinement begins with a 1,405 ft (428.24 m) tangent on the Washington approach. A 30 min curve left continues from just north of the Evergreen Highway for 3,423 ft (104.33 m). From this point a 825 ft (251.46 m) tangent joins a 1,000 ft (304.80 m) spiral which in turn connects to a one degree curve right that continues across the fill and South Channel structure. See Fig. 1. Later, to simplify alinement for the pre-cast segments, the spiral was reduced to a five curve compound curve.

TABLE 1.—Wild T16D Theodolite

Telescope (1)	Measure (2)	Erect image (3)
Magnification		30×
Clear objective aperture	1.65 in.	42 mm
Field of view at 1,000 ft/m	27 ft	27 m
Shortest focussing distance (multiplication factor)	5.6 ft	1.7 m
Additive constant		100
		0
Bubble sensitivity per 2 mm run:		
Circular level		8′
Plate level		30″
Automatic vertical index:		
Setting accuracy		±1″
Working range		±6′
Glass circles:	360° or	400^g
Graduation diameter Hz circle, V circle	3.70 in.	94 mm
	3.11 in.	79 mm
Graduation interval of Hz and V circles	1° or	1^g
Optical scale interval	1′ or	1^c
Estimation to	0.1′ or	0.1^c

FIG. 1.—Bridge Location with Alinement Data

TABLE 2.—Wild Distomat DI3S

Standard deviation (1)	±(5 mm + 5 × 10^{-6} × *D*), in meters (feet) switchable (2)
Distance measurement:	in meters or feet, switchable
Time from start to display	10 sec–12 seconds
Display (LED);	digital, six-figure, to mm or 0.01 ft
Slope distance unambiguous up to	999.999 m or 6,561.67 ft (= 2,000 m × 3.280835)
Reduction for horizontal distance, difference in height, difference in coordinates;	for distances up to 999.999 m or 6,561.67 ft
Angle input for reduction	six-figure (degrees, mins., tens of secs.), switchable for 360 or 400^{g}
Range under average atmospheric conditions:	
To single-prism reflector GDR31	≈ 1,000 m (3,300 ft)
To three-prism reflector GDR11	≈ 1,600 m (1 mile)
To nine-prism reflector GDR11 plus GDR2	≈ 2,000 m (6,600 ft)
Measuring scale factor, variable by scale change per switch step	11-step switch 3 mm/100 m, 0.03 ft/1,000 ft, i.e., $D \times 3 \times 10^{-5}$
Calculation time for reduction	0 sec–4 sec
Free objective aperture of emitting and receiving objectives:	35 mm
Focal length	38 mm
Emitting diode	GaAs luminescent diode
Receiving diode	Avalanche photodiode
Beam width at half power	4′ (120 mm at 100)
Carrier wave length	0.885 μm infra-red
Measuring scale frequencies:	
Fine measurement	7.4927 MH_z
Coarse measurement	74.927 kH_z
Emitted power:	about 0.02 mW
Power consumption:	
After switching ON and while measuring	≈ 17 W
When calculating and displaying	≈ 5 W
Small battery with built-in charger:	
Small battery, NiCd	12 V/1.8 Ah (10 × 1.2 V gas-tight cells)
Mains/line supply for charger	115 VAC or 220 VAC, 50 Hz–60 Hz
Time for charging flat battery	≈ 14 hr
Number of measurements at 20° C (68° F) with fully-charged battery	≈ 120
Large battery, NiCd, rechargeable:	12 V/7 Ah (10 × 1.2 V, gas-tight cells)
Number of measurements at 20° C (68° F) with fully-charged battery	≈ 500
Charging Unit Wild GKL 11, for large battery:	
Mains/line supply for charger	115/230 V ± 20%, 50 Hz–60 Hz
Temperature range of DI3S operation	−25° C–+50° C (−13° F–+122° F)

TABLE 2.—*Continued*

(1)	(2)
Dimensions (length × breadth × height):	
Aiming head (without counterweight)	190 mm × 110 mm × 70 mm (0.62 ft × 0.36 ft × 0.23 ft)
Control unit (without axis)	200 mm × 280 mm × 70 mm (0.66 ft × 0.92 ft × 0.23 ft)
Container (2 containers)	370 mm × 290 mm × 260 mm (1.21 ft × 0.95 ft × 0.85 ft)
Small battery	220 mm × 140 mm × 45 mm (0.72 ft × 0.46 ft × 0.15 ft)
Large battery	290 mm × 100 mm × 60 mm (0.95 ft × 0.33 ft × 0.20 ft)
Tilting range with aiming head on telescope	−65° to zenith (−70^g to zenith)
Control unit axis dish above tribrach dish	125 mm (0.41 ft)

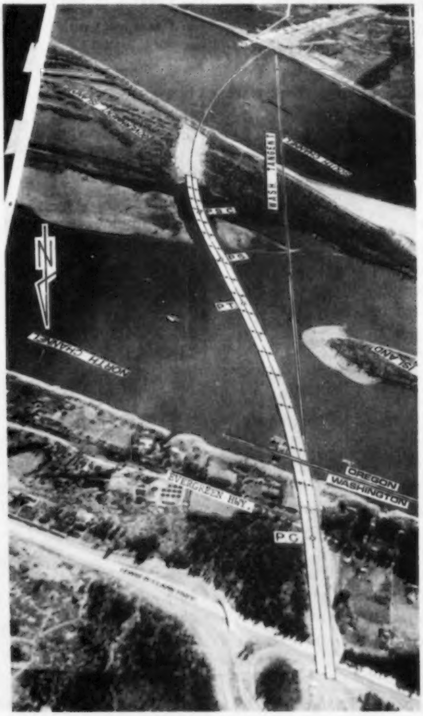

FIG. 2.—Washington Tangent Projected

Selection of Survey Equipment (Fig. 6)

Selection of the electronic distance measuring equipment was based largely on the recommendation of our location crew. With several years experience their Wild T16-DI3 combination seemed to give reliable results within the advertised range of 2,000 ft (900 m) and distances still looked good up to 4,000 ft (1,220 m). At this time the DI3 had just been upgraded to the DI3S(1) which has a capacity of 6,000 ft (2,000 m) which more than satisfied the one mile range needed at the site. The ease of aiming, converting slope distance and making atmospheric corrections were also factors in this selection.

The T16D(1) theodolite with the double graduated horizontal circle looked like an easy change for a crew experienced with the standard transit. This instrument reads directly to the nearest minute and estimation to the tenth of a minute (6 sec) is possible. This is obviously the weaker link and selection of this theodolite was based on the assumption that high order triangulation would not be necessary.

Preliminary Work

In January, 1977 our crew began establishing the line run by the location crew four years earlier. At that time they had projected ahead the Washington tangent, intersected it with the Oregon tangent, and found some discrepancies in both the angle of intersection and the length. See Fig. 2. Our first task with the Wild Distomat was to check this line and revise the alinement if necessary. The location crew had measured the length of the extended tangent with the DI3 and found it to be 9,944.22 ft (3,030.998 m). They checked it with a Hewlett Packard HP-3805 and obtained 9,944.34 ft (3,031.04 m). Our distance with the new DI3S was 9,944.33 ft (3,031.032 m). This not only confirmed the location work but gave us a great deal of confidence in the new equipment.

The most troublesome problem was the 17 sec error in the intersection angle of the two lines. With the center line run from both directions a divergence of up to 2 ft of the two lines would occur. It was decided to use the Washington alinement for control on the North Channel structure and the Oregon alinement for the South Channel. A revised alinement on the Government Island fill reconciled the difference in the two.

Construction of Permanent Monuments

For control during the 5-yr construction period it was our plan to build sheltered monuments on the shoreline outside the construction area. From the Washington shore the relative close proximity of Sand Island (2,200 ft, 670 m) and Government Island (3,500 ft, 1,070 m) eliminated the need for any survey stations in the river. The approximate elevation of Government Island is elevation 23, which is 4 ft higher than the mean river peak in June and looked like an acceptable elevation for the shelters. To give access to both sides of the twin structures the following five locations were selected. See Fig. 3.

Tower.—On the Washington bank approx 200 ft west of center line and 70 ft south of the USGS monument Gould. Bank elevation at this location was approximately elevation 9 so a low tower was needed.

Huron II.—On the Washington bank approx 1,000 ft east of center line and close to a former location traverse point.

Sand.—On the east end of Sand Island.

Tan.—On the projected Washington tangent and on the north shoreline of Government Island.

Bar.—On the shoreline of the dredged Government Island sand bar approx 400 ft east of center line.

Visibility of the structure from all points was good and from each location the other four points were visible. This permitted measuring all distances and angles and gave a tightly controlled geometric net.

All of the shelters were simple eight foot square plywood buildings. See Fig. 4. The windows were oriented so they gave an unobstructed view of the structure and the other four points. Separate foundations were provided for support of the instrument (Fig. 5) and in case of Tower a special attachment to the top of the vertical "H" pile was required (Fig. 6). At this location

FIG. 3.—Control Points and Traverse Nets

the monumented point was on the head of the pile (Fig. 7) and adjustment of the instrument over it in the usual manner was still required. The monumented points on Government and Sand Islands were 6 ft 8 in. section of 3 in. pipe augered and driven into the ground. Inside and around the top of the pipe was filled with concrete and a punched bronze disk set in the top for the point. At Tower a 4-ft square footing was excavated in the gravel formation; the H pile stood on end and concrete placed around it. This point has a tendency to vibrate if kicked and will move if leaned on. However, with careful use there is no problem, and results to date have checked with other points.

Traversing River

Work on the shelters as well as re-establishment of line by our crew was started in January and completed in April, 1977. This was in preparation for the May bid opening of the pier contract. In a three week period in April and May the distances were measured and the angles turned to the five points.

FIG. 4.—Shelter at Huron II

FIG. 5.—Monument and Support for Instrument Inside Huron II

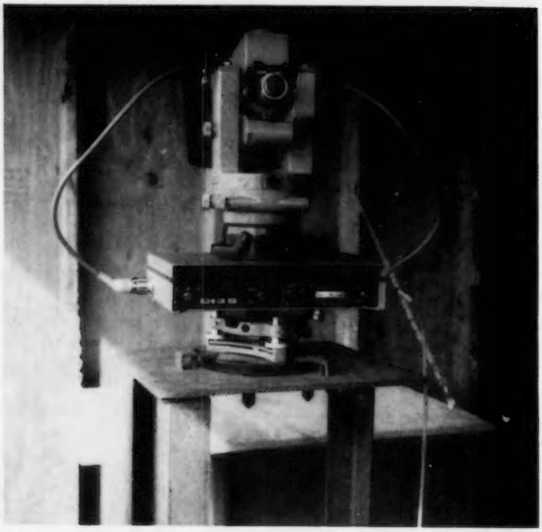

FIG. 6.—Instrument Setup at Tower. Trivet Clamped to Metal Plate Provides Base

FIG. 7.—Point in Head of *H* Pile at Tower

FIG. 8.—Single Prism Sight for DI3S which is used for Distances up to 3,300 ft

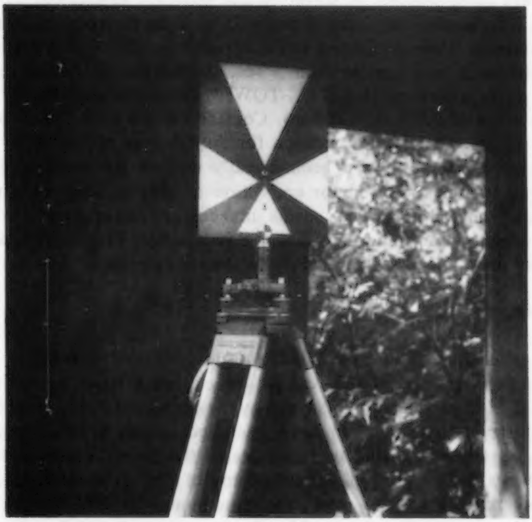

FIG. 9.—Sight Constructed for Turning Angles

Also, the points were tied into the P.C. located just north of the Evergreen Highway.

Distances were measured first, using two DI35 Distomats and the results averaged. The largest difference the two instruments showed was 0.02 ft; however, the weather seemed to have a more significant influence. A distance of 4,400 ft showed a difference of 0.04 ft when measured on sunny and rainy days. Slope distances were reduced in the field using the vertical angle measured with the T16D. Our maximum elevation difference in the five points was 4.32 ft (1.36 m) so slope corrections were 0.003 ft (0.0009 m) or less. Vertical control for the project was established at a later date using a tilting level circuit. ("Instructions for River Crossings," U.S. Department of Commerce Coast & Geodetic Survey Special Publication MO 239, Manual of Geodetic Leveling.)

After the distances were measured, all angles were turned twelve times. Sighting the target was the biggest problem and source of error. The single prism target used with the T16D-DI35 (Fig. 8) was found to be too small. Other commercial targets were investigated, seemed over priced and of a smaller size than desired. The end result is shown in Fig. 9 and was made by a crew member. This is a 9 in. × 12 in. piece of one quarter inch translucent plastic attached to a bracket with a steel pin that fits the prism mounting bracket. Inside the rather poorly lit shelters it was anticipated that a flashlight could be used from the backside to illuminate the target. This proved not to be necessary as the plastic seemed to have a light gathering property not typical of an opaque painted target.

Adjusting the traverse was largely a matter of judgement and utilizing the computer terminal that had access to the ICES COGO program. This program gives a very rapid solution of traverses and provides adjustment by the transit, compass, least squares or Crandall method. With the distances measured, angles for all triangles were computed and compared to the field measured values. If the difference was greater than 5 sec, the angle was turned again. Using the field data, traversing thru TAN-TOWER-HURON-SAND-BAR-TAN gave a closure error of 0.028 ft (0.008 m). Considering the total length of 12,127.29 ft (3,696 m) gives a relative precision for the traverse of 1:430000. The angles were adjusted so they were geometrically correct by comparing angles of overlapping triangles. The minor correction of lengths were then made using the Crandall method. This way seems unnecessary but it gives a good closure when traversing in any direction thru the five points. Finally coordinates were determined using the Washington P.C. as the initial point.

Conclusions

The high closure ratio leaves the question "is this possible?" and "Can continued performance of this caliber be expected from current day EDM equipment?" I think the answer is given by the continued check that has occurred during construction. The piers have been located by both triangulating and measuring distances utilizing two or more of the points. As a recent example Fig. 10 shows the location of a point on top of Pier 11 determined from three of the traverse points. The work was done January, 1980, approx 3 yr after the original survey, under cloudy skies that gave limited visibility. It was desired to accurately determine center line for maintaining alinement of the segmental

construction which was close to starting. Distances were measured and angles turned 12 times from the three shelters. The data were examined and it was decided to use the distances from HURON and SAND for control and discard the distance from BAR because of poor visibility. On Fig. 10 you will note the plot of the adjusted point gives a precise intersection of the two distances and three angles. Angular agreement in this instance is unusual and turning angles twelve times for most construction work on the project has not been common practice. In general, intersections on the project are determined by measuring the distances and doubling the angles from two of the points with the distance being favored as the more accurate. From the example it appears the monuments have not moved and the EDM equipment is continuing to give good results.

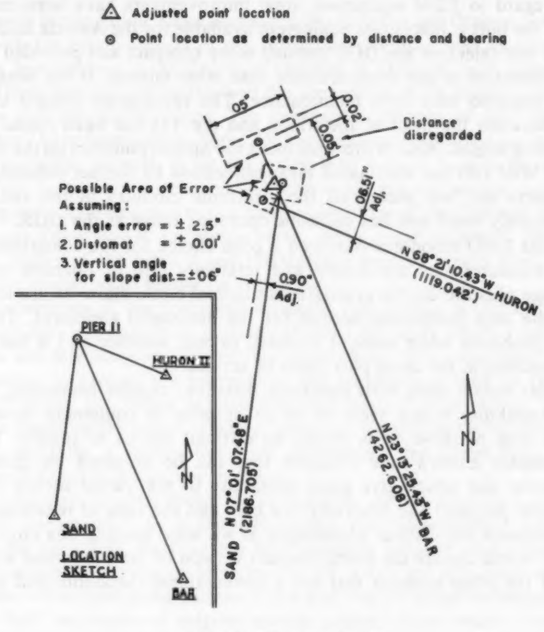

FIG. 10.—Ties to Pier 11

After more than 3 yr of construction work we feel our survey has served very well. Initially there were questions of where and how many monuments, what elevation and will they be clear of construction? How to construct them and are shelters needed to protect the instrument? The number of monuments more or less gets back to basics. Four points provide five ways of traversing the net and might be considered an optimum number. Additional points increase the number of angles to be turned, distances measured, and complicates closing the figure. In our case the fifth point on Sand Island brought all areas of the

structure to within a maximum distance of 1,200 ft (366 m) from one of the five monuments. The elevation of approx 23 ft (7.010 m) up to this time has remained above high water so was apparently a good choice. All points have been far enough away not to interfere nor be disturbed by construction activity. The long section of steel pipe plus concrete for the monuments may have been more than was needed; however, past experience with disturbed wood hubs and steel pins dictated something that would not move. Construction of the plywood shelters was a good decision. The total cost for the materials was approx $15,000 and they were built by the crew at a time when the work load was light. In addition to providing protection to the instrument from the strong east winds that occur in the area plus protecting from the sun and other elements, they further insure that the monuments will not be disturbed. The consistent accuracy that has been obtained would not be possible without these structures.

With regard to EDM equipment great improvements have been made since 1962 and the bulky, inaccurate equipment available for the Astoria Bridge. When we made our selection the DI3S seemed more compact and provided a simpler quicker reduction of the slope distance than other models. It has been accurate and has required very little maintenance. The rectangular control unit which mounts between the head of the tripod and the T16 has been found awkward when turning angles. Also, it prevents using the optical plummet on the theodolite. The new Wild DI4 has eliminated these objections by further reducing the size requirements and has placed all the electronic circuitry in the aiming head. This amazingly small unit has the same operating range as the DI3S.

The Wild T16D theodolite has been a good choice for the construction work. Angular measurements are simple and relatively free of operator error. The erect image plus the double graduated horizontal circle allow its use as a transit so it is the only instrument needed for the horizontal alinement. There have been no problems when using it exposed to wet weather and it has required no maintenance in the three plus years of service.

The total station units with electronic distance, angular measuring, and note keeping capability would seem to be an ultimate in equipment development. The high cost of these units would be difficult for us to justify. There are many compact EDM's now available that can be mounted on either transit or theodolite and which give great versatility to the varied survey jobs on a construction project. The relatively low cost and the ease of removing the unit for maintenance are distinct advantages. If we were starting this project today, the writer would choose the more compact version of the equipment we selected or one of the other systems that has a conventional theodolite and removable EDM.

Retracement Surveys in Pacific Northwest Coast Range[a]

By Gordon H. Long,[1] M. ASCE

Introduction

The United States Public Land Surveys in Washington and Oregon commenced in 1851 with the establishment of the Willamette Baseline and Meridian. "The Surveying Instructions to the Surveyor General of Oregon" was issued March 3, 1851. These instructions established the procedures to be used in establishing townships and their subdivisions. Ten subsequent editions of the "Manual of Instructions" have been issued. Some editions had significant changes in methods from the preceding manual. In all, the public land surveys have been performed under eleven manuals of instruction and their accompanying special instructions.

The original public land surveys on the western slope of the Cascade Mountains range in age from 70–130 yr. Retracement surveys are primarily 30–50 yr of age. The quality of the original and retracement surveys ranges from excellent to such low quality as to border on fraudulent.

The combination of the foregoing elements has created a challenge for the U.S. Forest Service, namely to properly identify the lands which it administers. The lands are intermingled with private ownerships. It is incumbent upon the forest service to protect the interests of the adjoiners as well as the federal land itself. Since the forest service does not have federal survey authority, reliance is placed on the state licensed practitioner to perform the needed surveys. Once the need for a survey has been determined, the forest land surveyor and his staff develop the survey package.

Under ordinary circumstances, an accelerated survey program can create a significant impact upon an agency. However, the forest service has handled the pressure very well. Success has been possible because of a corner search and remonumentation program implemented 20 yr ago and still continuing today.

Analysis.—The field-going personnel are well versed in the techniques needed to make field recoveries of original survey corners. Since most of the survey

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evidence on the western slope of the Cascades is badly deteriorated, extra care must be taken in searching for and evaluating the evidence. What is the evidence of the survey? The evidence of survey can be many things:

1. Monuments: A recovered monument, as established by the original surveyor, is the best evidence of a corner. The monument material used by the surveyor is usually local native material or what was readily available.

FIG. 1.—Pitchy Material Woodpost (United States Forest Service (USFS))

2. Posts: Woodposts 3 in.[2] by 4 ft were commonly used to mark the corner. Surveyors frequently state in their field notes the species of wood used. If he states that a redwood post was placed at the corner, the remnants of a sawn post may be found. If native material were used, it was cut on the spot and squared up to some extent. Usually, a pitchy material survives over the years. Young saplings rot away (Fig. 1).

3. Rocks: Instructions for setting survey corners called for durable stone with a volume of 1,000 in.[3] What was placed by the surveyor may approach this size. However, recovery of some corners will give an indication of his

reliability. If the field notes say notches and the stone found at the corner position is rounded and water-worn, grooves or scratches are more likely to be found. When the surveyor does not specify the setting of the stone to a depth, the possibility of the stone lying on the surface increases. The size of a stone in place may not be as given in the notes, due to sheet erosion or filling (Fig. 2).

4. Mounds and pits: Mounds and pits were often used to mark the position of a corner. A charred stake, charcoal, broken glass, or crockery was often

FIG. 2.—Grooves in Corner Stones (USFS)

used to mark the exact point. In time, the mounds are washed away and the pits filled. If there is some stony material in the soil, a large concentration of pebbles will be found at the location of the mound, and the pits will be filled with fine and wind-blown soil. The soil in the pits is not as compacted as the native soil and usually encourages better growth of vegetation. The material placed at the corner position stays in place and can usually be found.

Accessories to the Monument.—The information recorded in the field notes

FIGS. 3,4,5.—Bark Scribing (Gordon H. Long)

can actually be regarded as accessories since a definite relationship in bearing and distance can be determined. The value of the information is determined by the truthfulness of the surveyor's returns. These returns can be checked by a number of factors:

1. Bearing trees: The best accessory to the corner is the bearing tree. Early surveyors frequently miscalled the species. If the species were miscalled in

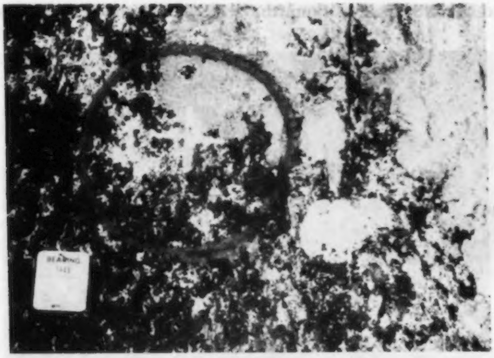


FIG. 6.—Survival Over Time of Bearing Objects (USFS)

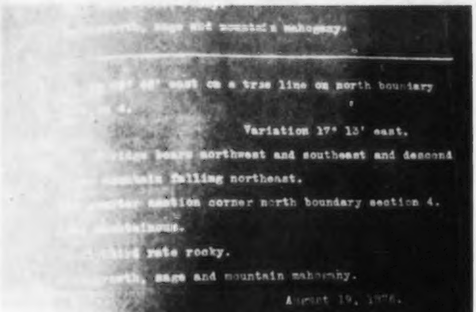


FIG. 7.—Topography Call Features (USFS)

one township, the surveyor probably repeated the error elsewhere. In large, old growth conifers, the surveyor would bark scribe instead of blazing. Bark scribing also was done to smooth the bark of deciduous trees. The platy bark of the conifers retains the original scribing, while on the deciduous trees, the scribing increases in width (Figs. 3, 4, & 5).

2. Bearing objects: Surveyors often would select bearing objects near a corner (a stone in place, cliff, ledge rock, or reef). Markings were cut into the exposed face. The bearing object stayed in place but the surrounding surface may have been altered by nature. The stone in place could be the top of a large boulder; the cliff face may scale, but the relationship of the object to the corner stays the same (Fig. 6).

3. Topog calls: Topography calls were given at the intersection of the natural feature with the true line. This, however, was not always done. The nearer the call is to the corner position, the less the likelihood of an error in the

FIG. 8.—Blazed Tree (Gordon H. Long)

relationship. A call for a rock ledge on line at six chains is more definable than a ridge-top at the same distance. The more distinctive the feature, the more definable it is (Fig. 7).

4. Use lines: The use line can be deceptive. Use lines are often established for convenience and not to show the limit of ownership. Old use lines, established shortly after the original survey, may be more reliable than those established in recent years. If the use line is supposed to be on the section line, walking it with field notes in hand and comparing the topography calls can determine

if it is of any value in locating the corner.

5. Blazes: Early surveyors had peculiar individual methods of making their blazes. The blazes met the GLO requirements on size and placement. Study of original blazes makes it possible to observe recognized individual styles. The age of the blaze can be estimated by cutting a small plug and counting growth rings. Another identifiable feature of authentic blazes is their following of a regular pattern (Fig. 8).

6. Pits and mounds: The pits and mounds were to be constructed in accordance with the special instructions. In grassland country, the work was usually done faithfully. In rough land, the pits were impracticable and a mound of stone would be constructed. A mound of stone can consist of as few as five stones or it can be skillfully laid of field stone and shoulder high. During a period of years, a stone mound will collect dirt and actually encourage the growth of brush on it. A scattered mound of stone can still be identified because of the concentration of stones in one spot.

7. Ties to auxillary points: Ties to auxillary points (mining claims, homesteads) can be useful if the tie is on the section line intersection with a claim line. A tie call must give a point tied to, e.g. "48.50 chns Corner 1 MS 955 bears N 73° E—5.56 chns," a call that can fix the position of a line better than "48.50 chns A building bears west approximately 18 chns." The former call is measured to a point; the latter is estimated and ill-defined.

Evaluation of Survey Evidence

Evaluation of survey evidence requires thoughtful comparison of the field note record and conditions on the ground. The evidence which is found—is it original? Is it an unrecorded perpetuation or is it where someone felt the corner should be? The habits of the surveyor provide the clues.

Feel for the Original Surveyor's Work.—The feel for a surveyor's work is developed through use of his field notes during the search. When the surveyor calls the size of the bearing trees, did he underestimate or overestimate the size? Does he call the direction of the ground slope every time? Are the notes full of detail but nothing can be found on the ground that resembles it? The truthfulness of his notes makes it possible to develop the feel of the surveyor's work (Fig. 9).

The technique of scribing and blazing is one index of a surveyor's work. The height of the blaze and the manner in which it was made is often sufficient to identify the surveyor. The tool used for scribing, was it fine or coarse? Was it jackknife or scribe? Were they Roman numerals or arabic? Was the top of the T an arc or a slash? This is the information that determines whether a blaze is original or not. A blaze on a tree always stays at the same height. A bark blaze stays at the same height, but the letters expand as the tree increases in diameter. A measuring notch cut at the crown of the roots many years ago can be buried by the accumulated duff and dirt. (Figs. 10, 11 & 12).

The System of measurement to bearing trees was not uniform among surveyors. Some measured from a measuring notch, some from the face of the blaze; others measured from the side center. Frequently, the surveyor measured on the slope without its being horizontal. Only by test measurement from original bearing trees can the system used by a surveyor be determined.

Applicable special instructions that the surveyor was issued concerned what manual he was to follow and the manner in which he would conduct his survey. These instructions make it possible to follow the footsteps of the surveyor. Copies of the manuals in use since 1851 should be in every library for reference as needed.

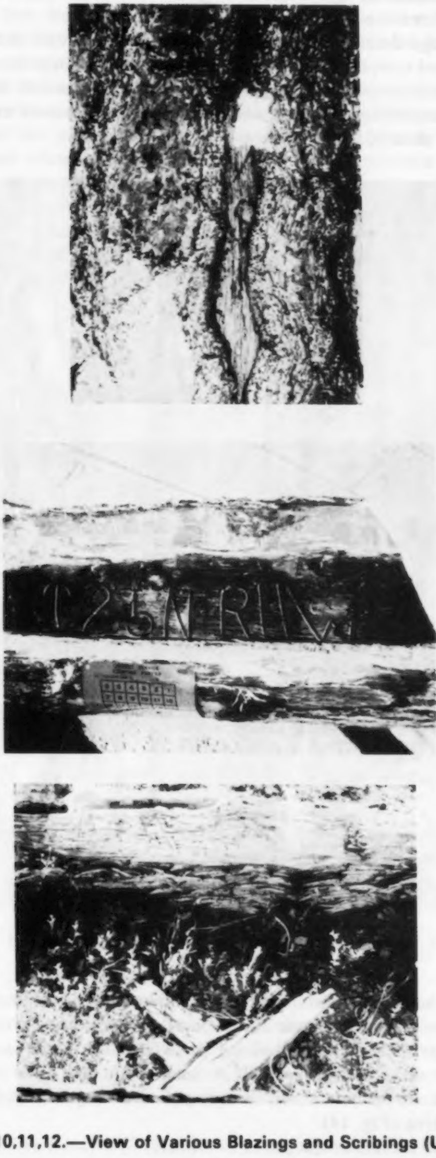
Bearing Trees as Evidence.—The surveyor was instructed to select hardy trees. In many cases, less than desirable species were used. In some cases, the species used was the climax species which no longer exists in the area. Care should

FIG. 9.—Surveyor's Original Blazing (Gordon H. Long)

be taken not to dismiss the notes as faulty without a thorough search for fossil remnants of the trees described.

During search and evaluation for corners, bearing trees should not be opened unless it is under the direction of a licensed land surveyor. The best method is to take on increment boring at the location of the blaze scar. The scribed surface should then be identifiable in the coring.

Bearing trees involves species determination. Species determination may rely on fragments of rotten wood, the smell of a freshly cut piece, or the visible cell structure under magnification. "Retracement and Evidence of Public Land



FIGS. 10,11,12.—View of Various Blazings and Scribings (USFS)

Surveys," published by the Eastern Region Forest Service, USDA, provides very useful information (Fig. 13).

To make an age determination, an increment borer is one of the better tools to remove a wood sample. Ring counting of overgrowth can give a fairly accurate date as to when the tree was blazed. Dyeing the sample makes the count easier. For very accurate dating of blazing and scribing, the services of a dendrochronologist should be secured.

FIG. 13.—Species Determination from Blazing (Gordon H. Long)

Many times, however, there is no surface indication of original bearing trees. The stump holes and patterns in the vicinity of the corner position can often be located by probing with a chaining pin. The soil is not as compacted in the stump hole as is the native soil. A small 1 in. soil tube sampler can be pressed into the stump hole and a sample pulled of any decayed wood without digging up the area (Fig. 14).

Reverse image scribes are at times necessary. Where a tree has been blazed,

a callus (overgrowth) will often develop. The woody material is often more dense or pitchy than the parent material. When the tree dies or rots, this material is more resistant. The reverse image of the scribing can be found as raised marks and letters on its surface.

The resolution of conflicting evidence requires the examination of the record notes and the evidence on the ground. The physical conditions described in the notes should be met as nearly as possible. These usually are the slope of the ground and other topographic calls. If the corner position does not agree with it, it is necessary to examine the evidence that its position has been based on. Many times an original corner stone has been moved and replaced in a different position because of an error in the transcribed field notes. In examining conflicts, it is best to use the original notes.

Parole Evidence (Testimony).—Information concerning a corner may be taken from local residents. The surveyor should have the resident state under oath that he had knowledge of the following facts: (1) What the corner looked like;

FIG. 14.—Excavated Decayed Wood (Gordon H. Long)

(2) the number of years he has resided in the area; (3) how he knew it was the corner; (4) when he was last at the corner; (5) the data; and (6) his name and age. This information should be entered in a field book. The statement should be signed by the person. In addition, the surveyor should enter a statement: (1) That the person was placed under oath; (2) that the statement was made by that person; (3) as to what the survey showed the person and what was placed there; (4) that the person was of good character and of sound mind; and (5) that the surveyor has signed and dated the statement. As soon as possible, the surveyor should file a corner affadavit with the proper authorities and include a copy of the statement as recorded in the field book. When a survey of record is filed it should also show the statement. (Recovery of the survey evidence does not accomplish anything unless it is perpetuated and made available for use.)

Perpetuation of Evidence.—Evidence may be perpetuated in many ways:

1. Documentation (field book, recordation certificate): A complete description of the recovered evidence is entered in a field book and on a corner recordation certificate. The information includes everything that was done and by whom.

2. Signing, painting, flagging are acts that can be done to draw attention to the evidence. The appropriate signs are nailed to the bearing trees, alive, dead, or down. The paint is used with care. The marks on the monument (stone or post) should not be painted over. Flagging should be tied abundantly to everything in the vicinity of the corner (Figs. 15 & 16).

3. New accessories: The original accessories to a corner are often in very poor condition. Sufficient supplemental accessories should be made that will aid in the recovery of the corner position. Marked Trees are one such accessory. After determining the location of the corner point, the measurement and the direction to selected references should be made. No less than two references

FIGS. 15,16.—Markings to Draw Attention to Evidence (Gordon H. Long)

should be taken. If a tree is used for a reference it should be marked in such a manner so that it cannot be confused with the original surveyor's mark. In the notes, indicate the point measured to Guard Posts are another. When a guard post is set at a corner, it should be set near, not at. Record the bearing and distance to it from the corner point. The bearing should be taken by standing at least ten feet from the corner point, looking through the corner point to the post. Finally, photogrametric targets are helpful. Frequently, aerial photo projects are scheduled in the same area that corner search is done. When possible, lay the proper size target at the corner. This will provide a recoverable point on the photography.

4. Photo identification: On existing aerial photography, a corner location can be photo-identified. Selection of an original bearing tree for identification is preferable. Other identifiable trees or objects near the corner can be chosen.

After making the photo-identification, a bearing and distance should be taken from the object to the corner point.

5. High stump BT: Many times a recovered BT is very decedent. When a chain saw is available it should be trimmed off above the blaze. The cut should slope downward sharply from the top of the blaze. The sloped surface will allow the moisture to run off and reduce the possibility of decay. Covering the cut with plastic, paint, or tar paper will help even more. High-stumping prevents wind-throw and untimely destruction of the bearing tree.

6. Reference monuments: The placement of reference monuments must be performed with care. The marking should be distinct and not easily confused with a corner marking. When two reference markers are set, they should be at 90° to each other. Do not drive a marker into the corner point location; for valuable evidence below the surface can occur.

Filing of Record Documents.—Recovery of corner point location and its perpetuation is not complete until the information is entered in the record. This involves three steps:

1. Corner record cards: The forest supervisor's office and the district ranger's office should receive and file a copy of the corner record card. A card is made out for all corners searched for, recovered or not.

2. Affidavits (testimony): The affidavit of parole evidence of a corner will be kept in a permanent file by the forest surveyor. The corner record card will have a notation as to the file and book number for reference.

3. Recordation forms: When a corner has been perpetuated based on parole evidence, the corner recordation form will have a copy of the parole affidavit attached. A copy of the corner recordation form will be filed with the county. Copies will be retained in a permanent file at the forest supervisor's office by the forest land surveyor. The district ranger's office will also maintain a copy. As a courtesy, a copy is forwarded to the state office BLM cadastral engineer.

These corner records are made available to the public as well as potential contractors. While relying on the corner records, the forest service also conducts a corner remonumentation program. This program work is done as part of a standard survey contract with a state-licensed surveyor, by the Forest Service Bureau of Land Management Cooperative Agreement, or by the forest land surveyor under his state license. By adherence to this practice, the forest service has been able to readily assemble the information needed to contract surveying services.

Conclusion

Retracement and subdivision surveys are being accomplished without any unusual difficulty. The corner search, evaluation, and monumentation program has relieved the surveying practitioner from much of the search for original survey evidence. This allows more time for the search for evidence of lost corners. An additional benefit has been that the proposals from potential contractors are based on known corner recoveries and not merely speculative

due to uncertain corner recoveries. In a three-fiscal-year period, 1977–1979, the Region Six Surveying Program has grown tenfold. It has been possible to commence surveys in areas that had previously been avoided prior to the search and remonumentation program. A mile of marked and posted national forest property line can require as much as two miles of survey line. The success of the Forest Service Surveying Program can be measured by accomplishment. In fiscal year 1979, 1,200 mi (1,932 km) of marked and posted property lines were surveyed in Washington and Oregon. The majority of the surveys were made on the western slope of the Cascade Mountains where the first surveys in Oregon began 129 yr ago. Recovery of the past speeds the surveyor today.

Acknowledgments

Several people have assisted me in developing the techniques described in this paper. Many thanks are due to my counterparts in the various Forest Service Regions; Stan Skousen, R1; Dave Branham, R2; Charles C. Doak, R3; and Walter Robillard, R8. I am in debt to the late Ira (Tiny) Tillotson for the "tricks-of-the-trade" which he willingly shared. My thanks also go to those with whom I work daily. Their interest in developing and supporting the land surveying profession is to be commended.

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Geometric Framework for Land Data Systems

By Kurt W. Bauer,[1] F. ASCE

Introduction

There is a growing interest in the United States today in the land data systems. This interest ranges from a relatively narrow concern about the need to modernize land title recordation systems to the relatively broad concern about the need to create entirely new land-related data banks for multipurpose application. This growing interest has involved many disciplines, ranging from surveyors, abstractors, assessors, and attorneys narrowly concerned with the fiscal and legal administration of real property to planners, engineers, and public administrators broadly concerned with community development and resource management. Much of the interest has centered around the use of electronic computers for the storage, manipulation, and retrieval of the data and, more recently, the use of graphic display hardware for the reproduction of the data in mapped as well as tabular form.

For practical reasons, the development of automated land data systems may have to begin with development of single-purpose cadastres relating to the registration of land ownership and perhaps to the value of real property as a basis for taxation. Such cadastres should, however, be amenable to evolutionary development into true land data systems that provide information on the characteristics, capabilities, and existing and potential uses of land for planning and management purposes as well as on the ownership and value of land.

Any land data system requires some method of spatial reference for the data. An adequate geometric framework for such reference must, if it is to serve even the narrower purposes of a cadastre, permit identification of land areas by coordinates down to the individual parcel level. The provision of a geometric framework of adequate accuracy and precision to permit system operation at a highly disaggregate parcel level is the most demanding specification possible and permits ready aggregation of information from the more intensive and detailed level to the more extensive and general level as may be necessary.

The decision concerning the type of geometric framework to be provided for any new land data system will be one of the key determinations affecting

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the long-term utility and efficiency of the system. Any error in this determination should be on the side of potential utility. A determination to provide a geometric framework more precise and accurate than may be required would ultimately mean that a part of the capital required to implement the system may be wasted. A determination, however, to provide a geometric framework less precise and accurate than may be required will ultimately mean that much, if not all, of the capital investment required to implement the system will have been wasted. Further, this capital investment itself may form an insurmountable impediment to later evolutionary development of the system since the committed decision will with time make it increasingly difficult and costly to effect any required reforms.

Because of the importance of the geometric framework for spatial reference of data to the long-term success of any multipurpose land data bank, and, because that importance is apt to be overlooked by planners and decision-makers in their deliberations of other important issues involved in the creation of land data systems, this paper discusses certain basic concepts that should be applied in the design of the geometric framework for any land data bank system. It also describes one kind of geometric framework based on these concepts and that can serve as a sound foundation for the evolutionary development of a multipurpose land data system.

Some Basic Concepts

A good multipurpose land data system must be able to store in coordinated, machine-readable form a wealth of data essential to sound land use planning and management. Historically, such data have been typically stored on maps. Consequently, certain concepts that apply to the design and preparation of good maps also apply to the design and implementation of the geometric framework for a land data system.

Any accurate mapping project requires the establishment of a system of survey control. This survey control consists of a framework of points whose horizontal and vertical positions and interrelationships have been accurately established by field surveys to which the map details are adjusted and against which such details can be checked. The survey control system should be carefully designed to fit the specific needs of the particular map being created. For multipurpose application, it is essential that this survey control system meet two basic criteria if the maps are to be effective planning and management tools. First, it must permit the accurate correlation of real property boundary line data with topographic, earth science, and other land and land-related data. Second, it must be permanently monumented on the ground so that lines on the maps may be accurately reproduced in the field when land use development and management projects reach the regulatory or construction stage. That is, the survey control system must provide finished maps, the points and lines of which not only accurately reflect both cadastral and earth science field conditions but also points and lines which can be readily and accurately reproduced upon the ground as well. This property is important not only to the use of the maps but to the maintenance of the maps in a current condition. Conceptually, the geometric framework for a land data system is the equivalent of the survey control system for a map; the same principles apply to its design and implementation.

Unfortunately, in the United States, two different, and heretofore largely uncoordinated, systems of survey control have evolved. First, the State Plane Coordinate System is founded in the science of measurement and utilized as a basis for the collection of earth science data and the preparation of earth science maps, such as topographic, geologic, soils, and hydrographic maps. The other—the U.S. Public Land Survey System—is founded in the principles of property law, as well as in the science of measurement, and is utilized for the collection of cadastral data and the preparation of cadastral maps, such as real boundary line maps.

U.S. Public Land Survey System.—For most of the United States, the federal government has provided the basic survey control system for cadastral mapping in the form of the U.S. Public Land Survey System. (Under the regulations imposed by the Congress, the U.S. Public Land Survey System has been extended into 30 of the 50 states, covering all but Connecticut, Delaware, Georgia, Hawaii, Kentucky, Maine, Maryland, Massachusetts, New Hampshire, New Jersey, New York, North Carolina, Pennsylvania, Rhode Island, South Carolina, Tennessee, Texas, Vermont, Virginia, and West Virginia.) This system is founded in the best features of the English common law of boundaries, superimposing on that body of law systematic land survey procedures under which the original public domain is surveyed, monumented, and platted before patents are issued; legal descriptions are by reference to a plat; lines actually run and marked on the ground control boundaries; adjoiners are respected; and the body of law in effect at the time of the issuing of the deed is controlling, and forever a part of, the deed. Unlike scientific surveys, which are made for the collection of information and can be amended to meet improved standards or changing conditions, the original government land survey in an area cannot be ignored, repudiated, altered, or corrected as long as it controls rights vested in lands affected.

The U.S. Public Land Survey system is one of the finest systems ever devised for describing and marking land. It provides a basis for a clear, unambiguous title to land, together with the physical means by which that title can be related to the land it describes. The system is ingenious, yet simple, easy to comprehend and administer; without it, the nation would be unquestionably poorer. The "rectangular" land survey system, however, has one serious flaw. Its use requires the perpetuation of monuments set by the original government surveyors, monuments the positions of which are not precisely related to the surface of the earth through a scientifically established map projection.

State Plane Coordinate System.—A strictly scientific survey control system designed to provide the basic control for all federal—and most private—topographic and other earth science mapping operations exists separately from the U.S. Public Land Survey System in the triangulation and traverse stations established by the National Geodetic Survey (formerly U.S. Coast and Geodetic Survey). The triangulation and traverse stations established by this agency comprise a nationwide network connecting thousands of monumented points whose geodetic positions in terms of latitude and longitude are known. In order to make the National Geodetic Survey control network more readily available for local use, the U.S. Coast and Geodetic Survey devised the State Plane Coordinate System in 1933. This system transforms the spherical coordinates—latitudes and longitudes—of the stations established in the national goedetic

survey into rectangular coordinates—eastings and northings—on a plane surface. This plane surface is mathematically related to the spheroid on which the spherical coordinates of latitude and longitude have been determined. The mutual relationship, which makes it practicable to pass with mathematical precision from a spherical to a plane coordinate system, makes it also practicable to utilize the precise scientific data of the national geodetic survey control network for the reference and control of local surveying and mapping operations. A limitation on such uses, however, is imposed by the relatively widespaced location of the basic triangulation and traverse stations and the difficulties often encountered in the recovery and use of these stations.

Recommended Geometric Control System

From the foregoing brief discussion of the U.S. Public Land Survey and State Plane Coordinate Systems, it is apparent that two essentially unrelated survey control systems have been established in the United States by the federal government. One of these—the U.S. Public Land Survey System—is founded in the legal principles of the real property description and location and was designed primarily to provide a basis for the accurate location and conveyance of ownership rights in land. The other—the State Plane Coordinate System—is founded in the science of geodesy and was designed primarily to provide a basis for earth science mapping operations and for the conduct of high precision scientific and engineering surveys over large areas of the earth's surface. Both systems have severe inherent limitations for use as a geographic framework for a land data system. By combining these two separate survey systems into one integrated system, however, an ideal system for the geometric control required for land data systems is created. This ideal system requires the relocation and monumentation of all U.S. Public Land Survey section and quarter-section corners, including the centers of sections, within the geographic area for which the land data system is to be created and the utilization of these corners as stations in second order traverse and spirit level nets, both nets being tied to the National Geodetic Data. The traverse net establishes the true geographic positions of the U.S. Public Land Survey corners in the form of state plane coordinates while the spirit level net establishes the exact elevation above mean sea level of the monuments marking the corners.

Such a system of survey control has the following advantages as a geographic framework for a multipurpose land data system:

1. It provides an accurate system of control for the collection and coordination of cadastral data since the boundaries of the original government land subdivision form the basis for all subsequent property divisions and boundaries. Thus, all subsequent legal descriptions and plats must be tied to the U.S. Public Land Survey System; and the accurate reestablishment and monumentation of the quarter-section lines and corners permit the ready compilation of accurate property boundary line data and the ready maintenance of these data in a current form over time. The data can be readily and accurately updated and extended since all new land subdivisions must by law be tied to corners established in the U.S. Public Land Survey and since the accuracy of the surveys for these subdivisions can be readily controlled by state and local land subdivision

regulations. The recommended survey control system, thus, fully meets the needs of a narrowly defined cadastre for the fiscal and legal administration of real property, yet a cadastre can be developed readily and soundly into a multipurpose land data system.

2. It provides a common system of control for the collection and mapping of both cadastral and earth science data. By relocating the U.S. Public Land Survey corners and accurately placing them on the State Plane Coordinate System, it becomes possible to accurately correlate real property boundary line information with earth science data. This placement of property boundary and earth science data on a common datum is absolutely essential to the sound development of any multipurpose land data system. Yet such a common control datum is rarely used. The establishment of state plane coordinates for the U.S. Public Land Survey corners permits the correlation with mathematical precision of data supplied by aerial and other forms of earth science mapping with property boundary line data compiled through the usual land surveying methods. Only through such a common geometric control system can all of the information required for a multipurpose land data system be accurately collected for, and correlated in, the system.

3. It permits lines and areas entered into the data base—whether these lines represent the limits of land to be reserved for future public uses, the limits of land to be taken for immediate public use, the limits of districts to which public regulations are to be applied, or the location and alignment of proposed new property boundary lines or of proposed constructed works—to be accurately and precisely reproduced upon the ground.

4. The system is readily adaptable to the latest survey techniques and is of relatively low cost as compared to alternative survey control systems that meet the criteria for use with a multipurpose land data system.

The specific geometric framework described here is applicable only to those parts of the United States covered by the U.S. Public Land Survey System. The fundamental concept involved—i.e., the need to place both cadastral and earth science data on a common geometric base—is, however, applicable to any area. In those portions of the United States that have not been covered by the U.S. Public Land Survey System, the application of this concept may well be more difficult and costly, requiring the incremental placement of the corners of the individual real property boundaries on the State Plane Coordinate System, but is just as essential if a comprehensive land data system is created over time.

Application

The recommended system of survey control has been to date widely utilized within the Southeastern Wisconsin Region, a seven-county, 2,689 square-mile planning area. One of the important reasons advanced for the application of this control system within the region was to prepare the way for the eventual development of an automated land data system which would have multipurpose application. To data, 4,891, or about 42%, of the U.S. Public Land Survey corners in the region have been relocated, monumented, and placed upon the State Plane Coordinate System in order to permit the development of large-scale

topographic and cadastral maps for 885 square miles, or about 33% of the total area of the region. This survey network already forms the basis for a regional planning data bank in which all forms of comprehensive planning data—both socioeconomic and geophysical—are related to a common geographic framework at the one-quarter section level. It permits the ready and economic creation of a modern land title recordation system readily adaptable to the application of interactive graphics techniques for the use of the data, all of which can be overlaid on a parcel level map. Once the cadastral and earth science data are stored in a computer, the data are scale free; and maps can be plotted by machine from the data at any desired scale. The coordinate values can be readily transformed from one system to another, if necessary; metrication can be accomplished by computer programming.

Summary and Conclusion

The development of dynamic land data systems that can be practically implemented initially as single-purpose cadastres providing information for the fiscal and legal administration of real property, but can also be evolved efficiently into multipurpose land data banks, providing information essential for comprehensive land use planning and management, will require careful planning and design. Such land data systems ultimately depend for successful application upon some method of spatial reference for the various kinds of data involved. Indeed, the geometric framework for spatial reference is one of the essential factors upon which the ultimate success of failure of any land data system will depend. The necessary geometric framework should permit identification of land areas by coordinates down to the individual parcel level while permitting the precise mathematical correlation of real property boundary and earth science data. By combining the two essentially unrelated survey control systems heretofore established in the United States by the federal government for real property boundary and earth science mapping, the necessary geometric framework can be provided. This requires the relocation and monumentation of all the U.S. Public Land Survey corners within the geographic area for which the land data system is to be created and the utilization of these corners as stations in second order traverse and spirit level nets tied to the National Geodetic Datum. The traverse nets establish the true geographic positions of the U.S. Public Land Survey corners in the form of state plane coordinates, thereby providing a common system of control for the collection and coordination of both cadastral and earth science data. The monumented, coordinated corners, in turn, provide the basis for readily maintaining the data base in current condition since future surveys can be tied to these corners.

The specific geometric framework described herein is, of course, applicable only to those parts of the United States which have been covered by the U.S. Public Land Survey System. The fundamental concept involved—i.e., the need to place both cadastral and earth science data on a common geometric base—is, however applicable to any area. In those portions of the United States that have not been covered by the U.S. Public Land Survey System, the application of this concept may well be more difficult and costly, requiring the incremental placement of the corners of the individual real property boundaries on the State

Plane Coordinate System, but is just as essential if a comprehensive land data system is to be created over time.

The importance of the establishment of a sound geometric framework for land data systems is apt to be overlooked by planners and decision-makers as a technical detail in their deliberations of other important issues involved in the creation of such systems. The establishment of a sound geometric framework for land data systems is, however, a fundamental, as well as a major, undertaking which clearly will require much understanding, foresight, and commitment on the part of the technicians and decision-makers concerned. Failure to make the proper decisions concerning this basic foundation of any land data system during the formative period will jeopardize the future utility of the system, for reform will become increasingly costly and difficult over time.

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DISCUSSION

Note.—This paper is part of the Journal of the Surveying and Mapping Division, Proceedings of the American Society of Civil Engineers, ©ASCE, Vol. 107, No. SU1, November 1981. ISSN 0569-8073/81/0001-0069/$01.00.

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The original and three copies of the Discussion should be submitted on 8-1/2-in. (220-mm) by 11-in. (280-mm) white bond paper, typed double-spaced with wide margins. The length of a Discussion is restricted to two *Journal* pages (about four typewritten double-spaced pages of manuscript including figures and tables); the editors will delete matter extraneous to the subject under discussion. If a Discussion is over two pages long it will be returned for shortening. All Discussions will be reviewed by the editors and the Division's or Council's Publications Committees. In some cases, Discussions will be returned to discussers for rewriting, or they may be encouraged to submit a paper or technical note rather than a Discussion.

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Note that the discusser's identification footnote should follow consecutively from the original paper/technical note. If the paper/technical note under discussion contained footnote numbers 1 and 2, the first Discussion would begin with footnote 3, and subsequent Discussions would continue in sequence.

Figures supplied by the discusser should be designated by letters, starting with A. This also applies separately to tables and references. In referring to a figure, table, or reference that appeared in the original paper/technical note use the same number used in the original.

It is suggested that potential discussers request a copy of the *ASCE Authors' Guide to the Publications of ASCE* for more detailed information on preparation and submission of manuscripts.

Definition of Mean High Water Line[a]

Discussion by Jack E. Guth,[3] J. F. Doig,[4] Steacy Hicks,[5] and John S. Grimes[6]

Based on his own Technical Note, *"Definition of Mean High Water Line,"* the author, Gunther Greulich, has asked for, and requested ASCE to publish, as a discussion, various definitions of the term "Mean High Water Line" from four contributors. The writers are Jack E. Guth, J. F. Doig, Steacy Hicks, and John S. Grimes.

Steacy Hicks offers. . . . The official definition of the National Ocean Survey, NOS, for the Mean High Water, MHW, Line is

> The intersection of the land with the water surface at the elevation of Mean High Water.

The official definition of the United States Government (through the statuatory authority of the National Ocean Survey to establish, define, and compute tidal data) for Mean High Water is

> The arithmetic mean of the high water heights observed over a specific 19-year Metonic cycle (The National Tidal Datum Epoch). For stations with shorter series, simultaneous observational comparisons are made with a primary control station in order to derive the equivalent of a 19-year value. For a semidiurnal of mixed tide, the two high waters of each tidal day are included in the mean. When any lower high water is indistinct, it is determined by record examination. For a diurnal tide, the one high water of each tidal day is used in the mean. In the event a second high water occurs, only the diurnal high water is included. So determined, this Mean High Water, based on the diurnal tide, is the equivalent of Mean High Water of a mixed tide.

The American Society of Civil Engineers (ASCE) wishes to have one definition (Mean High Water Line) which involves another major definition (Mean High Water). Also, in view of the probable desire of the ASCE to stay as close to their own wording as possible, the following is offered:

> The Mean High Water Line of a Tidal body of water is that line that

[a] November, 1979, by Gunther H. Greulich (Proc. Paper 14943).

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is made by the intersection of the sloped surface of the land with the water surface at the elevation of Mean High Water. Mean High Water is the arithmetic mean of the water heights observed for a series of points over a specific 19-year Metonic cycle. The specific 19-year cycle is known as the National Tidal Datum Epoch. For stations with shorter series, simultaneous observational comparisons are made with a primary control station in order to derive the equivalent of a 19-year value. For a semidiurnal or mixed tide, the two high waters of each tidal day are included in the mean. When any lower high water is indistinct, it is determined by record examination. For a diurnal tide, the one high water of each tidal day is used in the mean. In the event a second high water occurs, only the diurnal high water is included. So determined, this Mean High Water, based on the diurnal tide, is the equivalent of Mean High Water of a mixed tide.

Jack E. Guth states that

Through a recent State of New Jersey sponsored pilot study, using our instrumentation, techniques, and the EWE principal, we found that the system is so accurate we can now locate points of MHW where no landwater intersection occurs, where salt and fresh water meet, such as in coastal swamps and bogs, where heretofore it has been considered impossible to locate MHW points. In these areas where the tide gradually diminishes a MHW tidal influence could not be practically demonstrated. However, with our precise instrumentation we can document a minute rise and fall of tide to a few hundredths of a foot. This now enables us to locate MHW points, within reasonable limits, for tidal boundary line demarcation. The same techniques are utilized to determine the "head of tide" up rivers and streams where the MHW tidal influence diminishes and can be practically demonstrated with our system. This new system requires an elaboration on the accepted MHW line definition as stated in the NOS Tide and Current Glossary: "The intersection of the land with the water surface at the elevation of MHW." It should include "where no land-water intersection occurs it is at a point where the MHW tidal influence can no longer be practically demonstrated."

According to J. F. Doig the definition should be broken down into three basic components:

Style

The following might be considered more economical of words: The Mean High Water Line of a tidal body of water is the line of intersection of the sloped surface . . .

Technical Validity

Strictly speaking, the elevations read on a tidal gauge are valid for the location of the gauge alone. But it is a well established principle

that the gauge readings farily reflect tidal behavior in the immediate vicinity—with the outer limits of that "immediate vicinity" dependant upon circumstances peculiar to the locality.

Sooner or later in this day and age any definition which is laid down will be channenged as to its relevance or application to a particular set of circumstances. This being the case, is it wise to include the phrase "for a series of points?"

It would not be difficult to frame a number of questions which would show that a line adopted pursuant to observations at a series of points might well have a value not read at any one of them or which is not the mean of the values read at any one of them.

Faced with such a possibility, it might be wise to offer a definition which deals with one point only—a single tide gauge location as it were. Armed with this, the individual or an agency would have the freedom to compare or compile data from more than one location and be responsible to show that this treatment of the data was valid under particular circumstances.

Origin

Is it wise to tie the definition to the Borax case? Anyone at all aware of mean high water problems would recognize the definition as one which leaned very heavily on Borax for its authority, at the very least. To link the definition openly to Borax is an invitiation to distinguish between the Borax case and the particular case one has in mind now or at any other time in the future.

If the above points have come up in previous discusison and have been resolved in favor of the definition proposed, that is the end of the matter. If these points have not been so discussed, it might be worthwhile thinking about them for a bit.

In his letter of January 14, 1980, to Greulich, John S. Grimes states

> As respects tidal waters, MHW has four basically different rules: tidal ebb and flow, vegetation and typographical and public trust zone. Of these, I favor the typographical approach both in areas of surveying and boundaries. The ebb rule is, in my judgment, too indefinite and too subject to change with tidal variations.
>
> I realize that the vegetation rule has been used both in tidal and fresh water. This, however, in my mind is too difficult of application to be reliable. Vegetation changes too often during the year to be a proper indicator of a fixed boundary. Likewise, different plants flourish in various areas. I, thus, like the sloped surface even though subject to accretion, reliction, and erosion. The high water heights in the Borax Case at least furnish us with a line that can be pinpointed.

TRILATERATION ADJUSTMENT BY FINITE ELEMENTS[a]

Errata

The following corrections should be made to the original paper:

Page 73, first line in introduction: Should read "a plane structural framework" instead of "a plan structural framework"
Page 75, fourth line from bottom: Should read "azimuth" instead of "aximuth"
Page 77, two lines above Eq. 6: Should read "To overcome this difficulty, the unmeasured distance P_1P_4 must . . ." instead of "To overcome this difficulty, the measured distance P_1P_4 must . . ."
Page 81, Eq. 15*a*: Should read "(j) TYPE PLANETRUSS $E(L_j)$ AX (P_j)" instead of "(j) TYPE PLANETRUSS $E(L_i)$ AX (P_j)"
Page 92, Appendix I, Reference 3: Should read "3. Danial, N. F., "Adjustment of Trilateration Nets with Fixed Points," Joint Proceedings of the ASP-ACSM 1979 Fall Technical Meeting, pp. 66–83, Survey Review, No. 197, July 1980, pp. 313–326." instead of "3. Danial, N. F., "Adjustment of Trilateration Nets with Fixed Points," Joint Proceedings of the ASP-ACSM 1979 Fall Technical Meeting, Survey Review, pp. 66–83 (in press)."
Page 92, Eighth line from bottom: Should read "N_i = initial or known northing of point i;" instead of "N_i = initial or known easting of point i;"

PROPOSED MANUAL ON SELECTION OF MAP USES, SCALES, AND ACCURACIES FOR ENGINEERING AND ASSOCIATED PURPOSES: MAP AVAILABILITY—CHAPTER VI[b]

Errata

The following corrections should be made to the original paper:

Page 150, Paragraph 6, line 7: Should read "Division of Local Government" instead of "Division of Planning"
Page 155, under Colo.: Should read "2000 Arapahoe St., Denver, Colo. 80205" instead of "Colorado State Bank Bldg., 1600 Broadway, Denver, Colo. 80202"

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[a]November, 1980, by Naguib F. Danial and Theodor Krauthammer (Proc. Paper 15812).
[b]November, 1980, by Robert L. Brown (Proc. Paper 15856).